Nomination of Gladesville Bridge NSW

as an

Engineering Heritage International Marker

Addendum to
GLADESVILLE BRIDGE
Notes in support of Engineers Australia
***Engineering Landmark* nomination.**

Roads & Maritime Services, February 2014

Michael Clarke
for Sydney Engineering Heritage Committee
April 2014

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Contents

Introduction

Advice was received on 21 March 2014 that Engineering Heritage Australia's Heritage Recognition Committee had approved Gladesville Bridge as an Engineering Heritage National Marker (EHNM). That advice was based on notes prepared by the NSW Roads and Maritime Services (RMS) authority to assist the Sydney Engineering Heritage Committee with a nomination of Gladesville Bridge as an engineering heritage landmark.

The notes provide an excellent and detailed historical account of the design and construction of the bridge and of its social and technological significance, together with numerous images and interesting extracts from oral history. However, while the notes include most of the suggested content for a nomination, there is no formal assessment of the bridge's heritage significance or of a statement of significance, although there is a *Summary of achievements*.

As it is believed the bridge is of International significance, the case for upgrading the award to that of an Engineering Heritage International Marker (EHIM) is the subject of this Addendum.

Background to the design

In October 1955 Guy Maunsell (Attachment A) aged 71, the Senior Partner of an English consulting firm Maunsell, Posford & Pavry, established his own consultancy firm G. Maunsell & Partners. In order to obtain business Maunsell undertook a world tour looking for opportunities in the bridge and dock & harbour fields. While in Australia he became aware of the Gladesville Bridge proposal and formed the view that a concrete arch was likely to be a more economical solution than the steel cantilever designed by the Department of Main Roads (DMR).

As the tender documents allowed for alternative designs, Maunsell returned to England with a sketch of an arch bridge and arranged for a design to be developed to a stage where a contractor could tender on it. To avoid a large salary investment in the design Mr Anthony Gee, a 22 year old graduate was given the task of turning the sketch into a design that a contractor could price.

Meanwhile, Maunsell persuaded Reed & Mallik to team up with Stuart Bros of Sydney to put together a tender. Tenders closed on 17 October 1957, their tender was accepted and following extensive negotiations that included increasing the span from 977 ft span to 1,000 ft, the detailed design was put in hand (Attachment B).

Design of the bridge

Design and construction of the bridge is described in considerable detail in Attachment C.[1]

During assessment of the design the Department of Main Roads was advised by Professor Roderick of Sydney University and Eugène Freysinnet and his assistants in Société Technique pour l'Utilisation de la Précontrainte. The Consulting Engineers were advised by Professor A. J. Sutton Pippard, *M.B.E.,* D.Sc, F.R.S., M.I.C.E.[1]

The design of Gladesville Bridge set several new standards and contained a number of innovations in bridge design and construction that have subsequently been widely adopted.[2]

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[1] Attachment C: Gladesville Bridge, by John Walter Baxter, B.Sc. (Eng.), M.I.C.E., Anthony Francis Gee, M.A., A.M.I.C.E. and Howard Baikie James, B.Sc. (Eng.), A.M.I.C.E. Proceedings of Institution of Civil Engineers, 13 April 1965.

[2] Attachment D: civil + structural Engineer, http://www.gostructural.com/article/8682/gladesville-bridge-at-50.

The bridge and its design created such international interest by the engineering profession, that articles appeared in the following authoritative journals[3]:

- Engineering News Record: *Concrete arches to a world record*, 27 September, 1962;
- Engineering: *Record Concrete Span over the Paramatta* (sic*)*, 25 January 1963; and
- Civil Engineering and Public Works Review: *Constructing the New Gladesville Bridge,* December 1964.

Assessment of Significance

The Engineering Heritage International Marker *is intended for those works which have engineering significance beyond Australia.*

Criteria

a. Historical significance: *A work is important in the course, or pattern, of cultural history.*

Gladesville Bridge replaced the 1881 two-lane low-level lattice girder bridge that also carried trams and which had become a bottleneck on the major Victoria Road corridor.

The replacement bridge was built as part of the proposed North West Freeway along with the Tarban Creek and Fig Tree bridges. The bridges were part of a post World War 2 project to create a line of road linking Sydney City to the northern suburbs and through to Newcastle. However, the project was abandoned in the 1970s due to protests about the freeway's projected route through inner city suburbs such as Glebe and Annandale.

Gladesville and the associated bridges are thus evidence of both the road planning of the era that responded to burgeoning car ownership throughout Australia following the war, and the emergence of resident/citizen action groups who began to oppose major projects both public and private, that impinged on their quality of life, the environment and on heritage sites.

b. Historic Individuals or Association: *A work has strong or special association with the life or works of a person, or group of persons, of historical importance.*

Gladesville Bridge has a strong association with a number of eminent engineers and consulting engineers:

- Concept designs for the bridge were developed by the Bridge Section of the Department of Main Roads (DMR) headed by the Design Engineer Cliff Robertson who had been one of J J C Bradfield’s senior assistants on the Sydney Harbour Bridge. The Section also included a number of European-trained engineers, in particular steel bridge expert Vladimir Karmalsky and Albert Fried who later designed many pre-stressed concrete bridges. Brian Pearson, an eminent and highly respected bridge engineer was Supervising Engineer for the Gladesville Bridge and ultimately became Chief Engineer, Bridges.
- The DMR’s Bridge Section evaluated and accepted the alternative concrete arch design for the bridge by the London firm of consulting engineers G Maunsell & Partners, that was submitted in its tender by the English company Reed and Mallik, in association with an Australian firm Stuart Brothers.
- Design of the concrete arch bridge was by 22 year old graduate Anthony Gee in the office of G Maunsell & Partners. Gee later established consulting practices in both the UK and the USA. He has been responsible for the design and construction of major steel and

[3] Attachment E.

concrete bridges worldwide and is now Principal of Tony Gee International (Attachment F).

- Over time, Maunsells attained a global reputation including in Australia and Hong Kong, and won numerous contracts for new roads and highway infrastructure that included Europe's longest elevated concrete roadway, the Westway (1966-70). By the late 1990s the firm was the third largest civil engineering consultancy for transportation in the world.[4]
- Consultants to the DMR for Gladesville Bridge were eminent structural engineer Professor J W Roderick of the University of Sydney, and Eugène Freyssinet (Attachment G) and associates in Société Technique pour l'Utilisation de la Précontrainte, Yves Guyon and Jean Muller. Freyssinet is renowned internationally as a bridge engineer and the pioneer of prestressed concrete, a technique that transformed the design of bridge and concrete structures. His key contribution was in utilising high-strength prestressing wire which could counteract the effects of creep and relaxation, and to develop anchorages and other technology. Freyssinet also invented the Freyssinet flat jack which offers a wide range of uses when the application or control of high forces is necessary or deformations have to be compensated. Flat jacks were used in construction of Gladesville Bridge to lift the arches off the falsework. A list of Freysinnet's key achievements or collaborations includes Gladesville Bridge.

Gladesville Bridge has both a strong and special relationship with Australia's bridge engineers.

c. Creative or Technical Achievement: *A work is important in demonstrating a high degree of creative or technical achievement.*

Gladesville Bridge has a high level of technical and aesthetic significance. Its '*design reflects the growing awareness at the time that form should be as important as function. Structures were expected to harmonise with their surroundings, whether landscapes or cityscapes. Bridges had a new kind of aesthetic — elegant and slender continuous spans were achievable more readily in concrete than other materials'.*[4]

Gladesville Bridge is impressive and visually distinctive; it is graceful, having striking clean lines, slender columns and an elevated deck that provides a sweeping appearance to the whole.

The bridge is a landmark from the road, the river and public access points, and can be seen even from the Sydney Harbour Bridge 3.5 km away.

The pedestrian railings while being relatively suicide-proof, allow a clear vision of the surrounding landscape. When driving over the bridge the verticals of the railing become almost invisible so that there are superb views up and down the Parramatta River, especially to the city and the Sydney Harbour Bridge.

Gladesville Bridge, one of the longest concrete arch bridges in the world and Sydney Harbour Bridge, one of the longest steel arch bridges in the world, are two internationally significant bridges crossing the same body of water only 3.5 km apart, with Gladesville sitting in graphic contrast to its older companion.

[4] http://www.engineering-timelines.com/who/Maunsell_G/maunsellGuy7.asp

The overall excellence of the design, aesthetically and functionally, was widely acknowledged and led, in 1965, to the Civic Design Award by the NSW Chapter of the Royal Australian Institute of Architects for a “Work of Outstanding Environmental Design”.

The design of Gladesville Bridge set several new standards and contained a number of innovations in bridge design and construction that have subsequently been widely adopted.

- It was the first 1,000-foot span concrete bridge. The recent development of cable-stayed concrete bridges has resulted in this being almost commonplace but the first big cable-stayed concrete bridge, the Ricardo Morandi-designed Maracaibo Bridge in Venezuela, with spans of 770 feet, was an almost exact contemporary of Gladesville.
- It held the distinction of being the longest span concrete arch bridge in the world for nearly 20 years, until the completion of the Krk Bridge in Croatia in 1980. After nearly 50 years, it still ranks seventh.
- It was the first major concrete arch bridge built using precast segments, albeit with (un-reinforced) cast-in-place joints.
- It was the first major concrete arch bridge jacked at the quarter-points, rather than at the crown. Moreover, it was only jacked once, during construction, whereas the Krk Bridge, for example, was re-jacked twice more in subsequent years after completion.
- It was one of the earliest concrete bridges in which the deck was made structurally continuous for live load by the use of unstressed reinforcement contained in cast-in- place concrete between the precast girders over the piers.
- It was probably the first major bridge to rely entirely on the flexibility of concrete columns to accommodate longitudinal movements of the deck due to shrinkage, creep and temperature variation. The 2,000-foot-long deck contains only two expansion joints and is monolithic with the top of each pier. Concrete hinges were incorporated to increase the effective slenderness ratio of the columns, and hence their flexibility, and to prevent moments being transmitted into the deck by the flexure of the piers. The columns have a solid rectangular section only 2 feet wide so that the tallest ones, being approximately 110 feet high, have an effective slenderness (*kl/r*) ratio in excess of 133.
- It was almost certainly the first bridge to use piers of this kind constructed from precast segments. It was also one of the first, if not the first, major bridge to use integral abutments, later to become a popular method of eliminating the problems surrounding bearings and expansion joints at conventional abutments.
- It was undoubtedly one of the first bridges for which the design utilized a suite of computer programs for analysis and detailed design. These programs were written for the purpose, as there was no such thing as proprietary engineering software on the market at that time.[2]
- the use in Australia of very high strength concrete in reinforced concrete structures;
- Freyssinet flat jacks that lifted the arch off its falsework and the concept of movable falsework.
- a launching gantry that advanced from pier to pier to mount very heavy deck beams.

Prior to the introduction of pre-stressing, the strongest concrete used by the DMR in reinforced construction was 3,000 p.s.i. (20.7 MPa). Once confidence was gained that they could consistently produce high strength concrete of excellent quality, the minimum strength of concrete on all pre-stressed concrete bridges became 6,000 p.s.i. (41.4 MPa). This meant huge savings in materials and significant improvements in the aesthetics of designs.

Gladesville Bridge represents a major transition in NSW from steel bridge technology as represented by Sydney Harbour Bridge, to that of concrete, and especially prestressed concrete.

Experience in the construction of Gladesville Bridge greatly enhanced the professional knowledge and technical confidence of not only the DMR bridge engineers, but of others in the profession throughout Australia.

d. Research Potential: *A work has potential to yield information that will contribute to an understanding of cultural history.*

Gladesville Bridge is not significant under this criterion.

e. Social: *A work has strong or special association with a particular community or cultural group for social, cultural or spiritual reasons.*

Gladesville Bridge was an element in the North West Freeway project which was one of a number of projects in Sydney that gave rise to resident/citizen action groups who opposed major projects, both public and private that impinged on their quality of life, the environment and on heritage sites. It thus has a special association with the Hunters Hill community and heritage and conservation groups generally.

Gladesville Bridge has both a strong and special relationship with Australia's bridge engineers due to its size, design and method of construction, and in the increase of their confidence to produce large and complex structures.

f. Rarity: *A work possesses uncommon, rare or endangered aspects of cultural history.*

Gladesville Bridge is rare in that it was built to a design that is no longer favoured by road and bridge building authorities. It is the only concrete box section, arched road bridge in Australia, with the exception of nearby Tarban Creek Bridge, which is technically different in both its structural form and construction.

g. Representativeness: *A work is important in demonstrating the principal characteristics of a class of works.*

Gladesville Bridge is an outstanding representative of concrete, box section, arch bridges; not only for its size and complexity of its design and construction, it is unique in Australia, and it is recognised internationally as a significant bridge.

h. Integrity/Intactness: *A work is intact as built or has been modified, with a description of such repairs or modifications.*

Gladesville Bridge is maintained in excellent condition.

The roadway was widened in the 1970s to allow for greater traffic flow over the bridge, and along the heavily trafficked route of Victoria Road. The bridge roadway was increased from six lanes to eight by taking in some of the width of pedestrian walkways on either side.

Statement of Significance

Gladesville Bridge is of international heritage significance for the contribution it made to bridge design and construction by setting new standards, and its inclusion of a number of innovations that were subsequently widely adopted.

Gladesville Bridge was

- the first 1,000-foot span concrete bridge and was the longest span concrete arch bridge in the world when completed in 1964 (it remained so for around 20 years);
- the first major concrete arch bridge built using precast segments;
- the first major concrete arch bridge jacked at the quarter-points, rather than at the crown;
- one of the earliest concrete bridges in which the deck was made structurally continuous for live load by the use of unstressed reinforcement contained in cast-in-place concrete between the precast girders over the piers;
- probably the first major bridge to rely entirely on the flexibility of concrete columns to accommodate longitudinal movements of the deck due to shrinkage, creep and temperature variation;
- one of the first, if not the first, major bridge to use integral abutments; and
- one of the first bridges for which the design utilised a suite of computer programs for analysis and detailed design, the programs being purpose-written.

The bridge has associations with:

- the British consulting engineering firm of Maunsells which developed a global reputation, won numerous design contracts for roads and highway infrastructure throughout the world, and by the late 1990s was the third largest civil engineering consultancy for transportation in the world; and
- Eugène Freyssinet, the renowned bridge engineer and pioneer of prestressed concrete, and his associates in Société Technique pour l'Utilisation de la Précontrainte.

The bridge is an outstanding representative of concrete, box section, arch bridges.

Gladesville Bridge (a concrete arch) and its older companion the Sydney Harbour Bridge (a steel arch), are recognised as two internationally significant bridges. Crossing the same body of water only 3.5 km apart they provide a comparison of bridge technology as it changed over 30 years, that is of interest to engineers and the public alike.

From a national perspective, the bridge

- provides evidence of both the road planning of the post World War 2 era that responded to burgeoning car ownership, and the emergence of resident/citizen action groups to oppose major projects, both public and private that impinge on their quality of life, the environment and on heritage sites;
- has a high level of aesthetic significance, its design reflecting the growing awareness that form should be as important as function and that structures should harmonise with their surroundings;
- represents a major transition from steel bridge technology as represented by Sydney Harbour Bridge, to that of concrete and especially prestressed concrete;
- introduced into Australia the use of very high strength concrete in reinforced concrete structures;
- greatly enhanced the professional knowledge and technical confidence of bridge engineers throughout Australia.

- has an association with the eminent structural engineer Professor J W Roderick of the University of Sydney;
- apart from its size and the complexity of its design and construction, is unique in Australia;
- is rare in that it is of a design no longer favoured by road and bridge building authorities;
- is the only concrete box section, arched road bridge of its type in Australia; and
- reflects high standards in both design and construction which embodied high technical advancement.

Recommendation

It is recommended that Gladesville Bridge be awarded an Engineering Heritage International Marker because of its international significance in bridge design and construction, its setting of new standards, and its inclusion of a number of innovations.

Interpretation

Two panels and two plaques are proposed because the ends of the bridge are in different local government areas: the Gladesville or northern end in the Municipality of Hunters Hill and the Drummoyne or southern end in the City of Canada Bay; the locations are shown on the site map.

The main panel and ceremony point will be on a rise adjacent the north east end of Gladesville Bridge in Huntleys Point Reserve. There is an oblique view from there of the bridge, to the Sydney Harbour Bridge and to the tops of the towers of the Anzac Bridge. The panorama thus links the three major bridges in Sydney, bridges that were constructed about 30 years apart and which represent the state of the art in bridge technology at the time they were built. A walking path passes by the site of the panel.

The Huntleys Point plaque would be on the northernmost column adjacent the pathway along Huntleys Point Road.

The Drummoyne panel would be in Cambridge Road Reserve under the southern approach, with the plaque on the column closest Drummoyne Avenue.

Design of the panels is being prepared by Roads and Maritime Services.

Attachment A

G. Maunsell & Partners

G. Maunsell & Partners[1,2]

In October 1955, Guy Maunsell aged 71 who had been the Senior Partner of an English consulting firm Maunsell, Posford & Pavry, established a consultancy firm G. Maunsell & Partners that eventually attained global recognition.

For the first three years of operations there were few UK contracts, possibly because Maunsell and his partners focused on quality of service rather than profitability, but this dearth was compensated for by plenty of work overseas, particularly bridge projects in Australia.

The company's first bridge in Australia was the Narrows Bridge (a National Engineering Landmark) over the Swan River in Perth, for which it was appointed consulting engineers in April 1956. In the same year it was also awarded the design contract for the Tasman Bridge over the Derwent River in Hobart.

After becoming aware while in Australia of the Gladesville Bridge proposal, Maunsell formed the view that a concrete arch was likely to be a more economical solution than the steel cantilever designed by the NSW Department of Main Roads. As the tender documents allowed for alternative designs, Maunsell returned to England with a sketch of an arch bridge and arranged for a design to be developed to a stage where a contractor could tender on it. To avoid a large salary investment in the design, Mr Anthony Gee, a 22 year old graduate was given the task of turning the sketch into a design that a contractor could price.

Maunsell persuaded Reed & Mallik to team up with Stuart Bros of Sydney to put together a tender. Tenders closed on 17 October 1957, their tender was accepted and following extensive negotiations that included increasing the span from 977 ft span to 1,000 ft, the detailed design was put in hand.

Work on site began in December 1959 and the bridge opened to traffic on 2nd October 1964. The design reflects the growing awareness at the time that form should be as important as function. Structures were expected to harmonise with their surroundings, whether landscapes or cityscapes. Bridges had a new kind of aesthetic, that elegant and slender continuous spans were achievable more readily in concrete than other materials.

Gladesville Bridge was Maunsell's last major project in Australia; sadly, he did not live to see it completed. He retired officially in 1959 suffering ill health, and died in 1961.

Maunsell's resolve and determination in the face of adversity served him well during his lifetime. After his death the company won numerous contracts for new roads and highway infrastructure, including Europe's longest elevated concrete roadway, the Westway (1966-70), and established a global reputation in Australia and Hong Kong. By the late 1990s the firm was the third largest civil engineering consultancy for transportation in the world.

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1 http://www.engineering-timelines.com/who/Maunsell_G/maunsellGuy7.asp

2 Attachment B.

Attachment B

Email Anthony Gee to Michael Clarke
22 April 2014

From: Tony Gee
Date: Tuesday 22 April 2014 6.32 AM
To: Michael Clarke
Subject: Re: Gladesville Bridge
Attach: (W) Gladesville Bridge at 50.docx (21.1 KB) (PDF) Gladesville articles (3.60 MB)

Michael,

Thank you for your e-mail. I am delighted to hear that the Gladesville Bridge might be getting an Engineering Heritage International Marker. I hope your nomination is successful.

I think it is fair to say that I was the design team. One thing is certain - there is nobody around to dispute this! John W. Baxter was the Partner-in-Charge but in all honesty his technical contribution was minimal.

I am sure you will be asking yourself how a 22-year old got the opportunity to design such a bridge so, although it has no bearing on the Heritage award, let me bore you with a little background. Guy Maunsell was the Senior Partner of a firm called Maunsell, Posford & Pavry and in 1955, following a dispute with Messrs. Posford and Pavry, he took 3 senior engineers (whom he made partners), 2 engineers and 3 draftsmen away to form G. Maunsell & Partners.

In order to drum up business, the "Old Man" as he was called, took off round the world looking for opportunities in the bridge and dock & harbour fields. While in Australia, he stumbled across the upcoming Gladesville Bridge and his unerring engineering instinct told him that a concrete arch was very likely to be a more economical solution than the steel truss the MRD had designed. Whether they had already decided to allow contractor alternatives or whether he persuade them to do so, I don't know.

Anyway, he came back to England with a pencil sketch and a rendering (he was an accomplished artist) of an arch bridge and told the Partners he wanted to have a design taken to the stage where he could get a contractor to put in a bid. I suspect the Partners thought that while the Old Man might not be completely off his rocker, it was an extremely long shot on which they were not prepared to invest a lot of scarce resources. I had been the firm's first graduate recruit in 1956 and as such was the cheapest resource available so I was deployed to work up the Old Man's sketch into something a contractor could price. And apart from a draftsman, I was given no assistance.

Meanwhile the Old Man persuaded Reed & Mallik to team up with Stuart Bros. to put together a price. When the bid was submitted, I believe it showed a substantial saving over the lowest price for the steel truss (or any other alternatives submitted). There followed an extended period of negotiations during which the Department, having apparently got a cheaper bridge than they had anticipated (and being Australian), asked how much it would cost to stretch the 977 ft span on which the tender was based to 1,000 ft for no other reason than they could have the first 1,000 ft concrete bridge. So it was back to the drawing board for a short time for me to produce the very slightly greater quantities. Apparently the increased price was acceptable and a provisional contract was signed - provisional because at that time the design was very much preliminary and there were only a handful of drawings. It was in fact probably more like Letter of Intent.

Anyway, this is where I have to cut a long story short and say that to their credit the Partners did not have the nerve to take my baby away from me and I was allowed to complete the detailed design. Again, I think it is fair to say that the groundbreaking features of the bridge described in

the article to which you referred were of my doing, as were the computer programs. When I tell my engineering acquaintances over here that I used a computer for bridge design in 1958, they flat out just don't believe me: the first recorded example of such a thing in America was in 1965 but fortunately I have 2 magazine articles to prove it!

After the detailed design was completed and submitted to the MRD for their approval, they decided that in view of the unprecedented nature of the design, they would obtain the services of a third party to review it and appointed Europe Etudes, the design arm of the Freyssinet organization, as the independent review authority. One of the most exciting aspects of my participation in the project to a young engineer, apart from my two visits to Australia, were the meetings with Eugene Freyssinet himself. Yves Guyon and their then relatively youthful protégé Jean Muller.

Incidentally, G. Maunsell & Partners changed their name after Gladesville to Maunsell & Partners because too many people were writing Gee, Maunsell & Partners!

As far as the technical features of the bridge are concerned, most of them are referred to in my article. One I forgot was the use of what the Americans call Bulb-T girders in the deck. Since they were introduced here in the 1980's, they have been heralded as the greatest thing since sliced bread.

You probably have access to more references than I have but I am attaching copies of some magazine articles of the time. I am also attaching my original manuscript of the recent article: it has a few additional facts which were 'edited' out.

Don't hesitate to get back to me if you need any more information.

Regards,
Tony Gee

From: Michael Clarke
Sent: Tuesday, April 15, 2014 8:14 PM
To: GEE, Tony
Subject: Gladesville Bridge

Dear Mr Gee
My name is Michael Clarke and as a member of the Sydney Engineering Committee I am preparing a nomination for the award by Engineering Heritage Australia of an Engineering Heritage International Marker for Gladesville Bridge, to coincide with its 50^{th} anniversary on 2^{nd} October.

While the design of Gladesville was by the firm of G Maunsell & Partners, I would like if I can, to identify the actual design team of which I believe you were one, and possibly the leader. It would be much appreciated if you could help me with this and possibly include comment on the development of the design and issues of special significance. In doing so, could you point me to any specific references relating to the design?

I have read your excellent description of the bridge and its significance (*Gladesville Bridge at 50)* at http://www.gostructural.com/article/8682/gladesville-bridge-at-50 ; this will be a very important source for the nomination as it really establishes the bridge's international significance.

With kind regards and thanks in anticipation
Michael Clarke

Attachment C

Gladesville Bridge,

by

John Walter Baxter, B.Sc.(Eng.), M.I.C.E., Anthony Francis Gee, M.A., A.M.I.C.E.

and

Howard Baikie James, B.Sc.(Eng.), A.M.I.C.E.

Proceedings of Institution of Civil Engineers, 13 April 1965.

(John Walter Baxter was Maunsell's Partner-in-Charge of the design,

Anthony Francis Gee was the designer, and

Howard Baikie James from the UK, was the

contractor's Chief Engineer or Agent).

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For discussion at an Ordinary Meeting on Tuesday, 13 April, 1965 at 5.30 p.m., and for subsequent written discussion

Paper No. 6869

GLADESVILLE BRIDGE

by

John Walter Baxter, B.Sc.(Eng.), M.I.C.E.

Anthony Francis Gee, M.A., A.M.I.C.E.

and

Howard Baikie James, B.Sc.(Eng.), A.M.I.C.E.

SYNOPSIS

Built for the Department of Main Roads, New South Wales, the new bridge at Gladesville provides a high level crossing of the Parramatta River for the new North Western Expressway serving Sydney. The structure consists of a reinforced-concrete arch spanning 1000 ft and supporting prestressed-concrete [illegible] columns and a prestressed-concrete beam deck of [illegible] spans. Footpaths 6 ft wide are provided on both sides of the bridge and the carriageway width is 72 ft, widening at the northern end to accommodate a slip road. Navigable clearance during construction was 80 ft high × 200 ft wide and the permanent clearance provided is 120 ft × 200 ft. The arch consists of four ribs 20 ft wide and 14 ft deep at the crown, built of hollow concrete voussoirs placed upon falsework, jointed with in situ concrete, and made into arches by means of flat jacks permanently built into the arch at the quarter points to induce thrust. The ribs were built and jacked in turn to levels above their true funicular shapes by amounts calculated to allow for future creep and shrinkage movements. The ribs were stressed together through transverse diaphragms at 50 ft centres. The falsework consisted of large universal trusses in high-tensile steel supported on steel tubular columns founded upon steel piles driven to or grouted into sandstone. The prestressed-concrete deck units were made on site adjacent to the bridge but all other precast units were made at a site downstream and brought to the bridge by water. The Paper covers the design of the principal components of the bridge and of the falsework and includes a full description of the problems of construction and the methods adopted and the plant employed to overcome them.

INTRODUCTION

THE comprehensive system of expressways being built by the Department of Main Roads, New South Wales, to serve the Sydney metropolitan area includes one through the north-western suburbs which crosses the Parramatta

Written discussion should reach the Institution before 15 May, 1965 and will be published after September 1965. Contributions should not exceed 1200 words.

FIG. 11.—VIEW OF STONE HARBOUR

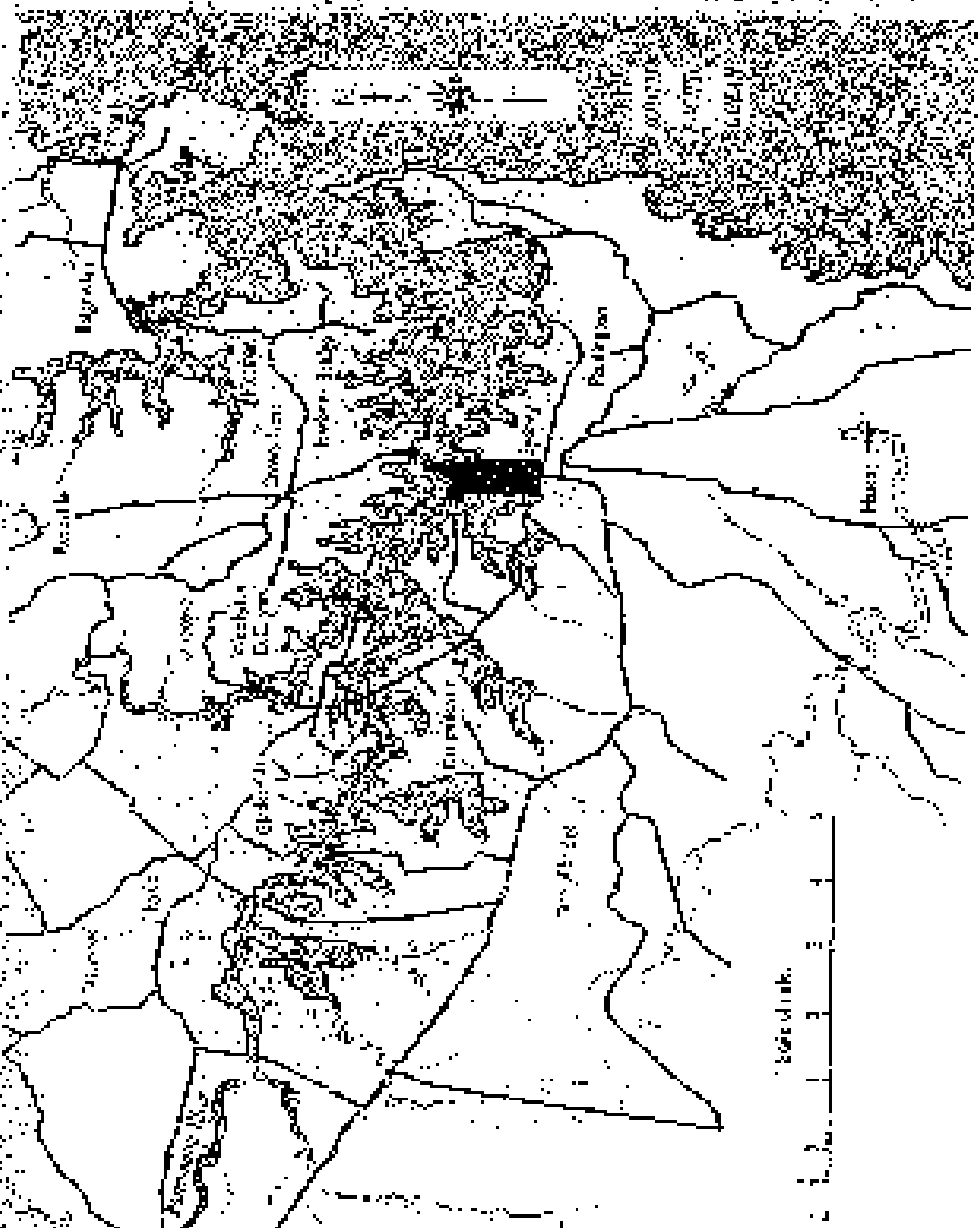


FIG. 2. MAP OF SYDNEY HARBOUR

Fig. 9. The completed Gladesville Bridge

River between Gladesville on the north bank and Drummoyne on the south (Fig. 2). The site is about 3½ miles west of Sydney Harbour bridge, close to the point where the river joins the harbour. The new bridge replaces a two-lane multi-span swing bridge.

2. In 1957 the Department of Main Roads invited tenders for the construction of a steel bridge of cantilever design prepared and fully detailed by the Department, and also, if tenderers wished, for the design and construction of alternatives. One alternative submission was a concrete arch design and this was thoroughly checked by the Department, assisted by the Professor of Civil Engineering at Sydney University and his staff. In the course of the investigation of the design certain changes were called for, agreed upon and adopted. On being satisfied that the proposal was sound the Commissioner for Main Roads decided to adopt the alternative. Subsequently a variation of the original alternative design involving a revised construction procedure was proposed. M. Freyssinet and his assistants in Société Technique pour l'Utilisation de la Précontrainte were engaged to report on certain aspects of this design. The recommendations made in this report were also agreed to and the Commissioner decided to proceed with the work in accordance with the revised alternative proposal. At all subsequent stages the Department of Main Roads kept a close watch on all aspects of design and construction. The Authors wish to express their appreciation of the objective decision to adopt and build the concrete arch.

Site conditions

3. At Gladesville the Parramatta River is about 1000 ft wide and has a maximum depth of 55 ft. The bedrock is Sydney sandstone which is exposed on the banks but overlain by clay and mud up to 90 ft in thickness in much of the river bed. The difference between mean high water and mean low water level at spring tides is 4½ ft.

Loading and stresses

4. The bridge was designed to carry a uniformly distributed load of 64 lb/ft run in each of the six lanes, plus a knife edge load in each lane, taken as 18 000 lb for bending or 26 000 lb for shear calculations. The local effects of a 72-ton truck and trailer were taken into account and the deck was checked for special loads of 75 ton in each of two lanes with other lanes carrying the standard loading. A reduction of 10% was allowed if three lanes and 25% if four, five or six lanes were considered loaded. Concrete design stresses were limited to 2000 lb/sq. in. in direct compression and 2250 lb/sq. in. in compression due to bending. An increase of 25% on these figures was permitted in the case of stress due to temperature.

GENERAL DESCRIPTION

5. The principal and dominant member of the bridge is a reinforced-concrete arch spanning 1000 ft clear across the Parramatta River. This arch supports prestressed-concrete piers which in turn support the deck structure of the bridge. Other piers are carried on the river banks and the deck structure finishes at each end in reinforced-concrete box abutments retaining rockfill

FIG. 4. ELEVATION OF BRIDGE

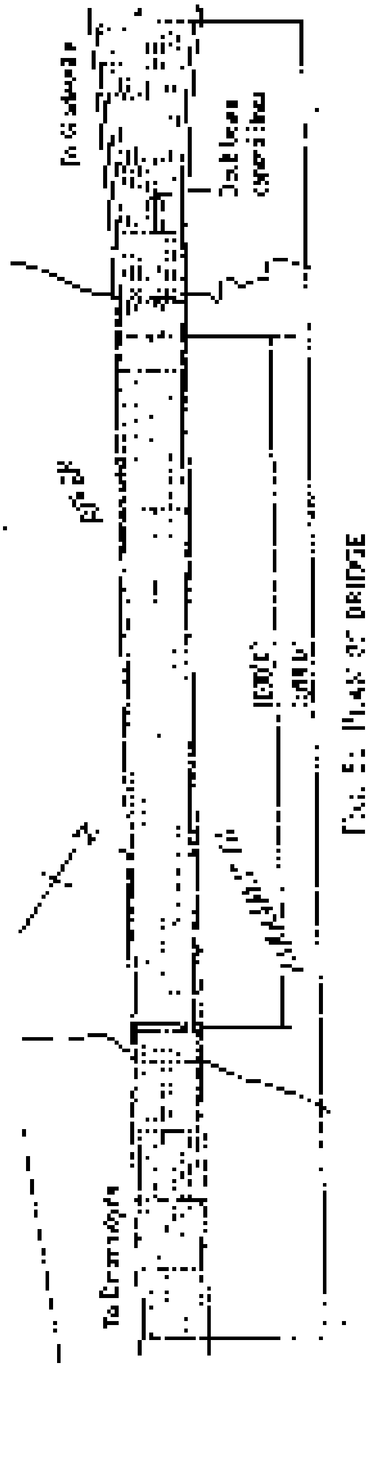


FIG. 5. PLAN OF BRIDGE

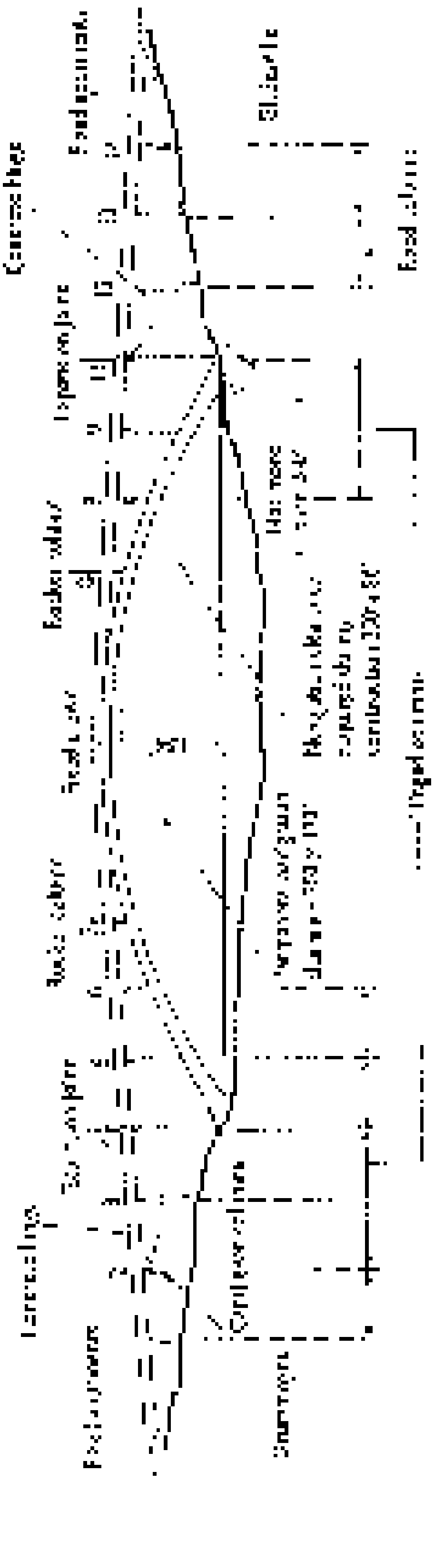


FIG. 6. ARTICULATION DIAGRAM

approach embankments. The centre section of the deck, 300 ft in length, is supported directly upon the arch but elsewhere the deck consists of 100-ft spans—eight on each side of the river. Three piers are founded on each bank, one on each arch abutment, and six on the arch itself. Each pier consists of two columns joined by a crosshead.

6. The bridge provides two walkways 6 ft wide and a six-lane carriageway 72 ft between kerbs generally but increasing in width at the Gladesville end where slip roads commence. A transition curve 300 ft in length at the crown of the bridge connects approach gradients of 6%.

Articulation

7. The eight deck spans on each side of the bridge are articulated as shown in Fig. 6. By adopting this system advantage was taken of the fixity available in the abutments and in the arch itself and, although there are only two expansion joints in the deck, the deck movements are related to monolithic lengths of only 400 ft.

Foundations

8. All foundations are of reinforced concrete bearing on hard sandstone at varying depths below ground level. In no case did the depth of excavation exceed 15 ft.

9. The approach embankments were placed by the Department of Main Roads after the construction of the reinforced-concrete deck abutments formed by counterfort type retaining walls stabilized by a wide heel and a small toe.

Piers

10. The supporting piers to the deck spans are in the form of two plate-like columns joined by a crosshead of the same thickness (Figs 7 and 8) to form a portal frame designed to resist all lateral forces. Longitudinal forces are transmitted back to the abutments or to the crown of the arch by the deck and the deck movements are accommodated by flexure of the columns whose lengths, by means of the articulation system chosen, are approximately proportional to the movements imposed.

11. The stresses induced in the columns by the longitudinal movements and horizontal forces are large compared with the axial stresses and, because of the very high slenderness ratio, it was deemed advisable to ensure the [illegible] of the columns by prestressing out all tensile stresses. This consideration did not apply to the crossheads where prestressing would have been unsuitable as the resulting shortening would have induced very high bending moments in the columns due to their comparative rigidity in the transverse direction. For this reason the crossheads are of heavily reinforced concrete. A detailed analysis of the columns was carried out in order to establish exactly their flexed shape and so calculate with some certainty the separate and collective effects of vertical load, horizontal deflexion, and initial lack of straightness, as well as the factor of safety against instability.

12. The columns are stressed by Macalloy bars which were wrapped in 'Denso' tape before casting. This method of construction eliminates ducts

Fig. 17. View in arch [illegible]

and grouting but prevents the development of bond between the steel and the concrete. The lack of bond is not in this case a disadvantage as it is not necessary to achieve a high ultimate bending strength. In the taller piers prestressing was carried out in stages to eliminate movement of the bar couplers.

213. A column designed in this way represents a compromise between the need for flexibility to accommodate deck movement and the need for stiffness to avoid buckling. It follows that a wide choice of dimensions is not possible and the columns which are flexible enough for their purpose are not stable as free-standing cantilevers, and require to be propped during construction. By using the two-stage prestressing for the longer columns and nominal reinforcement, one prop per column proved sufficient.

214. In order to speed construction the piers to be supported by the arch were precast in three large pieces and joined by prestressing together while the arch was being built. The very short piers nearest to the crown of the bridge incorporated a rudimentary hinge in the base of each column so that these could function effectively as rockers.

215. An important consideration in the design of the deck was the avoidance of undue weight as the point loads transmitted by the deck structure constituted a significant factor in the design of the arch. The deck spans generally

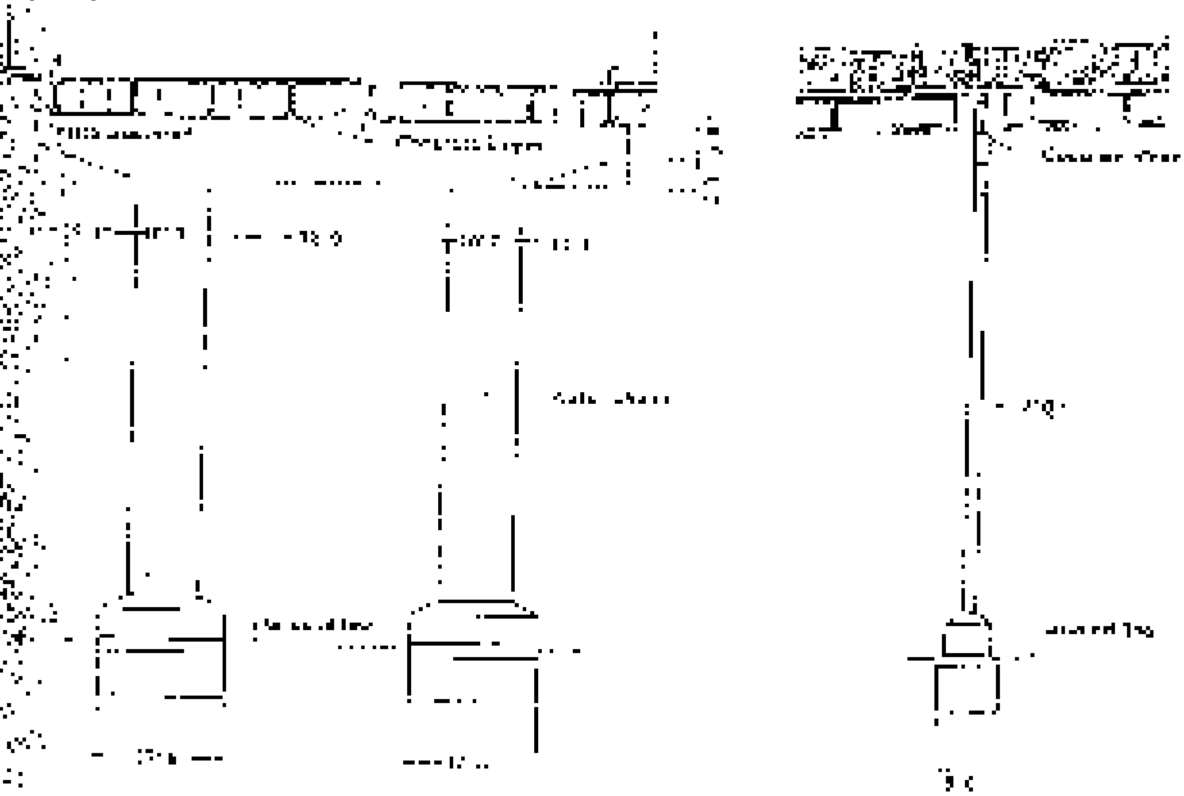


FIG. 8. COLUMN DETAILS

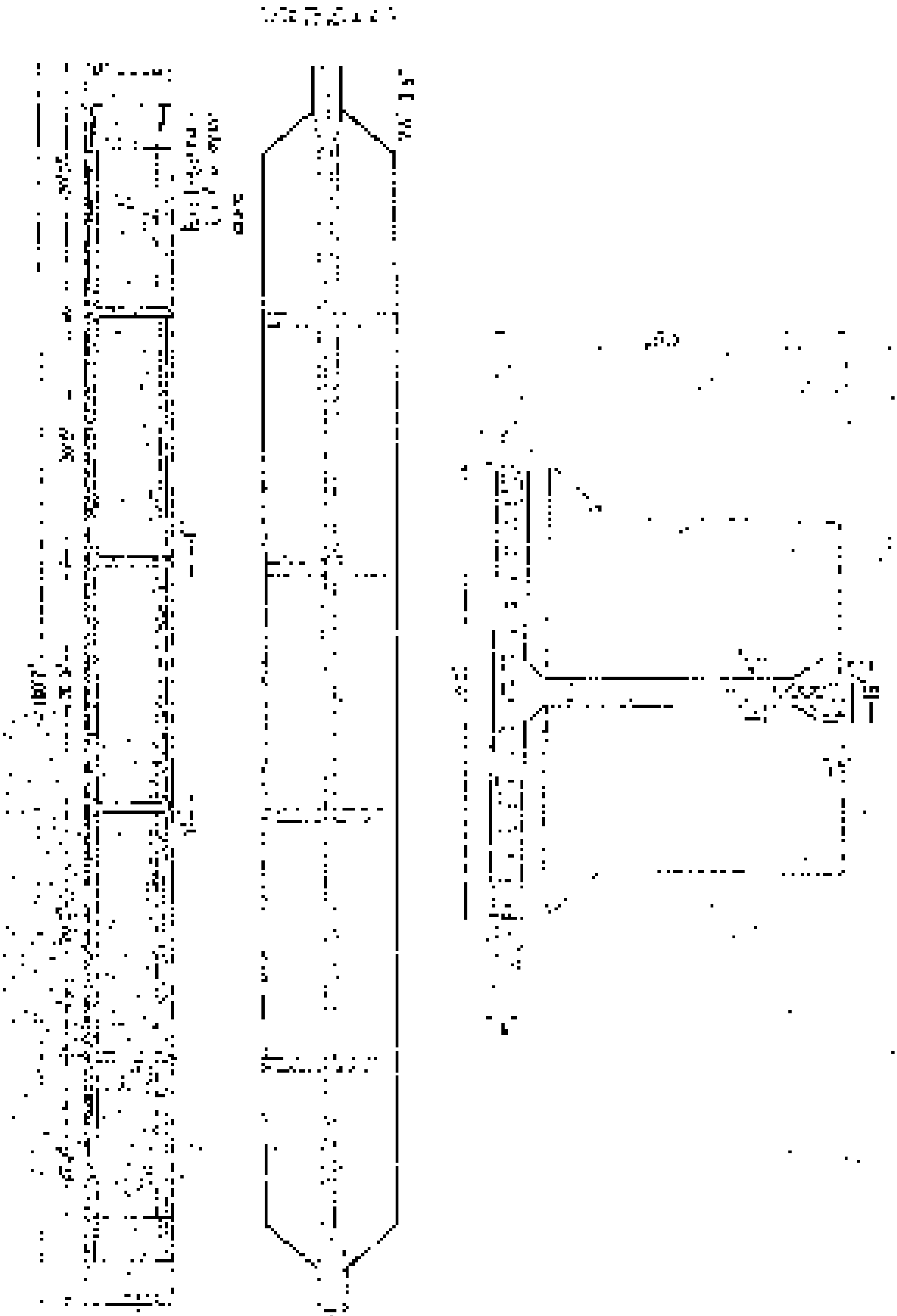


FIG. 9: DETAILS OF BEAM

comprise eight precast T-beams. Each beam weighed about 65 ton and was prestressed by only four cables using the P.S.C. twelve ½-in. dia. strand system (Fig. 7). The top flanges and projecting slab diaphragms of the beams were joined together by in situ concrete to form a complete beam-and-slab grillage. This was an awkward constructional detail, but the effect of the abnormal load on the almost square spans created a difficult design problem and it was considered an advantage to avoid transverse prestressing. No in situ concrete was placed over the precast beams themselves.

FIG. 10(a): [illegible]

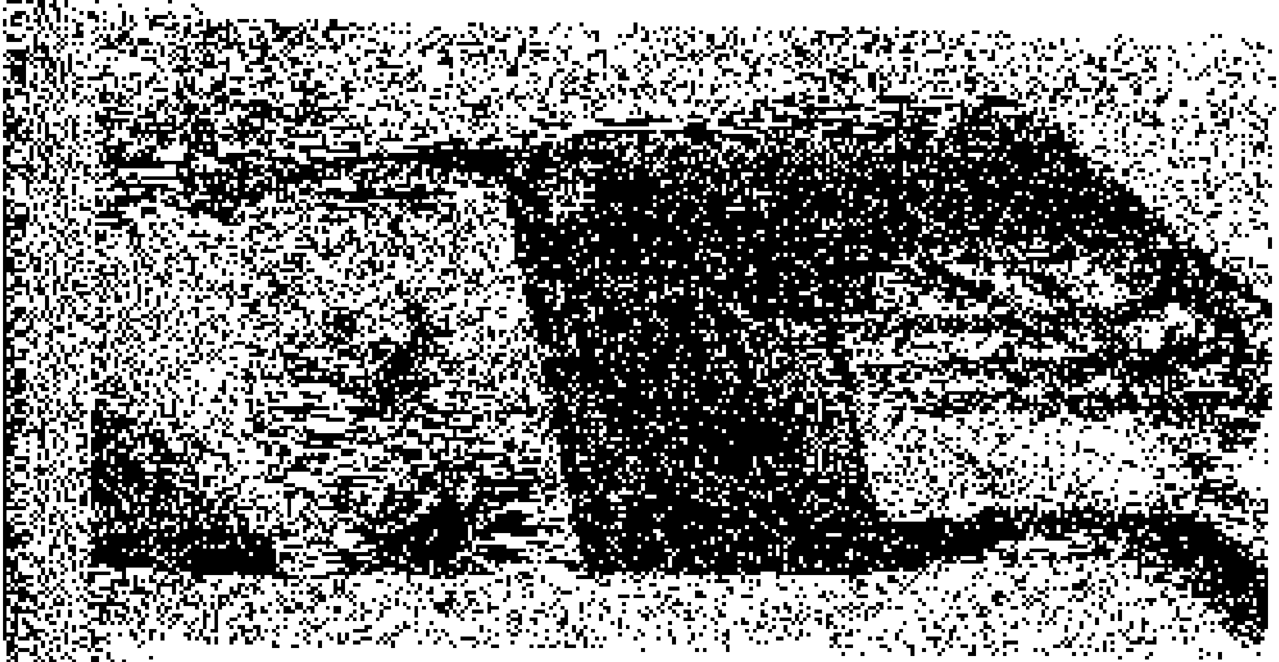


FIG. 10(b): [illegible]

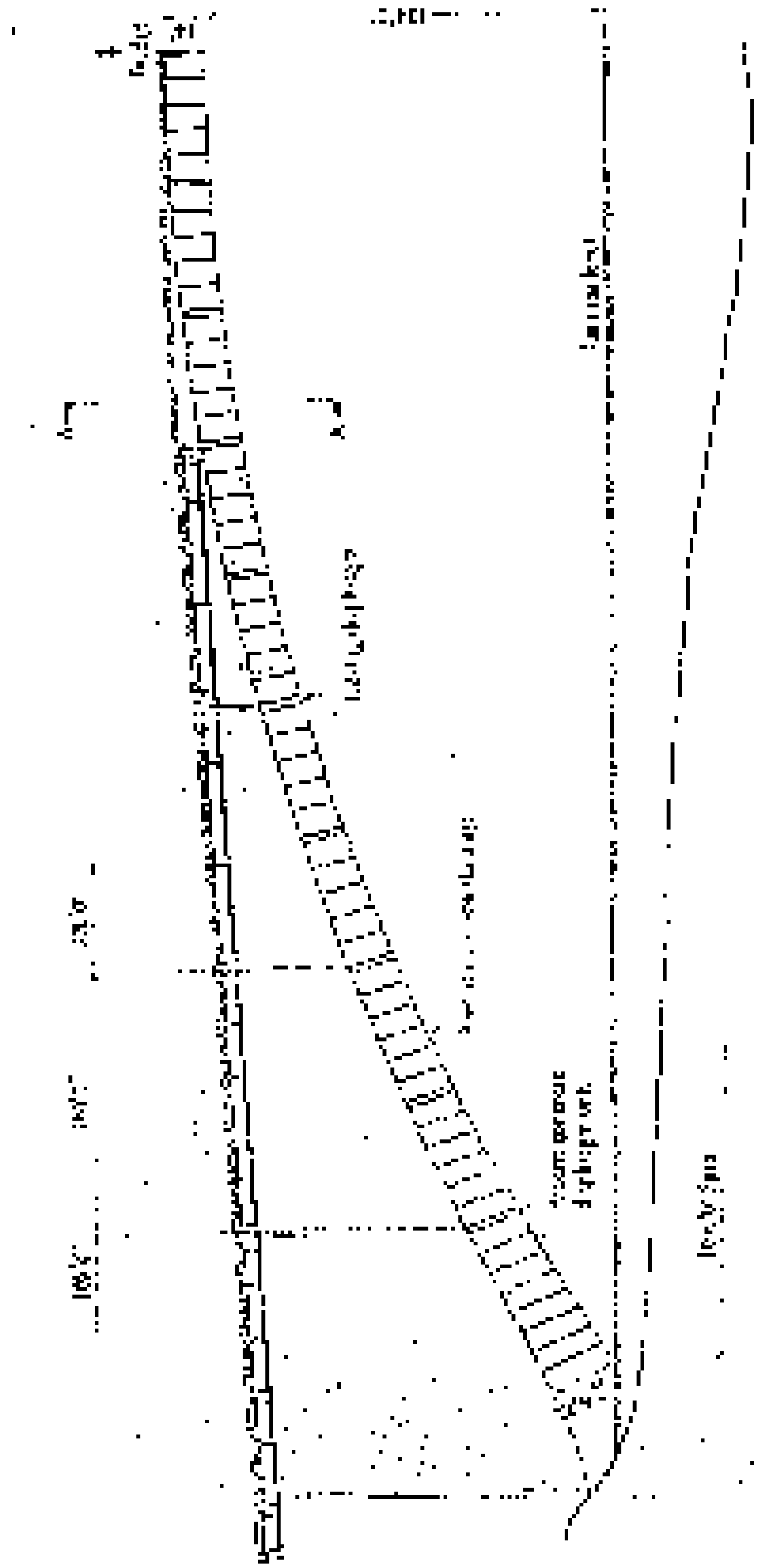


FIG. 17. ELEVATION OF ARCH

17. Longitudinal continuity was obtained by tapering the top flanges of the precast beams adjacent to the end-blocks, thus making room for sufficient in-situ concrete between each beam over the piers in which to include the longitudinal reinforcement necessary to cater for the live-load moments.

18. At the expansion joints halved joints were provided in the beams incorporating roller bearings (Figs 10(a) and 10(b)) which permit deck movement caused by temperature, creep, and shrinkage.

19. After the general principles of the design had been formulated, a decision was taken to include a traffic interchange between the Gladesville and the bridge and the proposed Tarban Creek Bridge adjacent. This necessitated widening the carriageway on the bridge itself over the last four spans at the Gladesville end. The increased width required was achieved by modifying the standard deck beams, retaining the basic shape but reducing the top flange width. These beams were placed closer together and the increase in width of each span was obtained by tapering the in situ portion of the deck slab between the precast beam flanges. Each of these smaller beams contains three prestressing cables and there are respectively 10, 11, 12, and 14 beams in the last four spans.

19. The beam supports had to be designed to provide hinges for the accommodation of longitudinal movement. At both ends of each beam a small precast concrete hinge was placed in a recess left in the crosshead and beams were supported temporarily at their correct levels whilst in situ diaphragms were cast between the end blocks.

20. As the end blocks of adjacent beams were not in line in the four spans at the Gladesville end, the hinge detail adopted elsewhere was not practicable. Fortunately these piers are slightly longer than the corresponding piers on the Drummoyne bank and with their greatly increased width occasioned by the wider deck above, it was found to be just possible to accept the longitudinal movement in flexure without a hinge although very heavy prestressing was necessary.

Arch

21. The arch (Fig. 11) was constructed as four separate hollow ribs which were later made monolithic by joining the diaphragms and the top flange

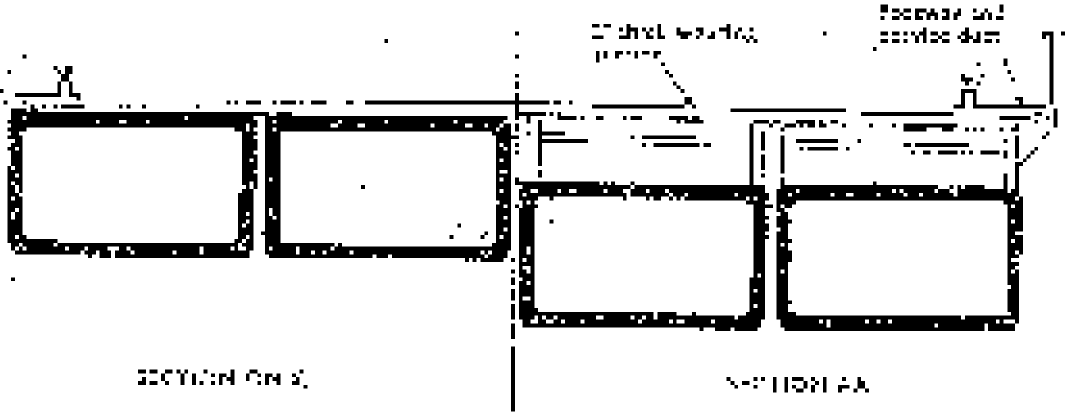


FIG. 12: CROSS-SECTION OF ARCH

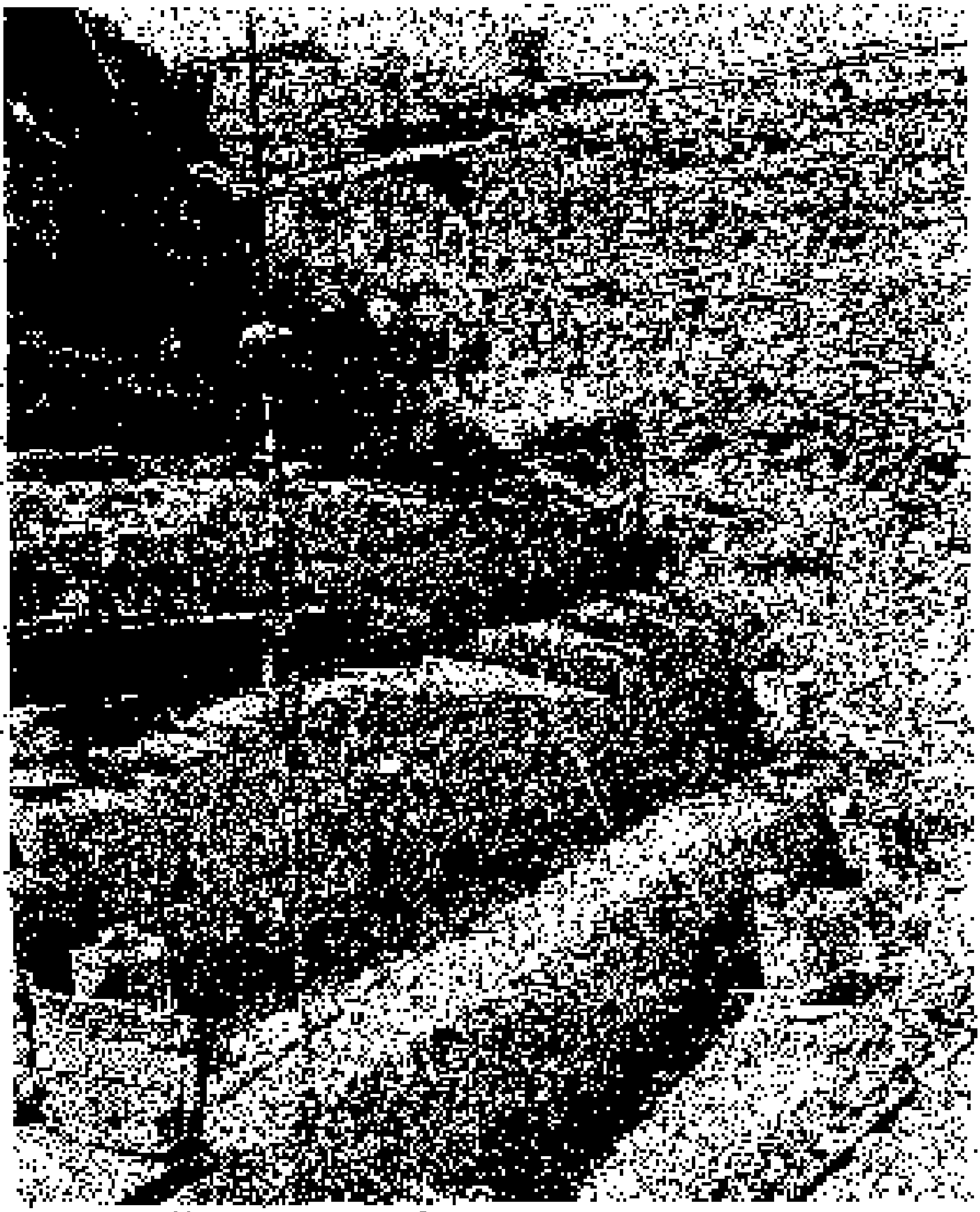


FIG. 17. ARCH ABUTMENT EXCAVATION

All the dimensions of the cross section of the ribs except the depth (Fig. 12) are constant throughout the length of the arch. The flanges are 15 in. deep and the webs are 12 in. thick. A more efficient section could have been achieved by increasing the flange thickness and reducing the webs but heavier reinforcement would have been required to resist self-weight stresses during construction and a reduction in the web thicknesses would not have been admissible near the springings where the units are 22 ft in height.

22. The arch abutments are of mass concrete founded directly on to hard sandstone (Fig. 13). Some 40 boreholes were sunk at the sites of the two abutments and in addition numerous probes were made. The depth of excavation actually carried out was controlled by the testing of cores taken from the rock as work progressed but the final limits of excavation were remarkably close to the estimated profiles shown on the drawings and on which the theoretical bearing pressures were calculated. A corridor runs across inside each abutment giving access to the interior of the four ribs and this is entered through a trap door in the roof of the abutment (Fig. 14).

23. The central part of the bridge deck, 300 ft in length, is supported directly on the arch. The deck slab is carried on a series of parallel transverse walls which were cast in situ and located by reinforcement projecting from the joints between the arch rib units. Precast slabs spanned between these walls and acted compositely with the in situ topping concrete. For a distance of about 50 ft on either side of the centreline the slab is cast directly on to, but insulated from, the arch ribs. In this way, the actual stiffness of the arch over this length was virtually unaffected by the deck structure. All longitudinal forces derived

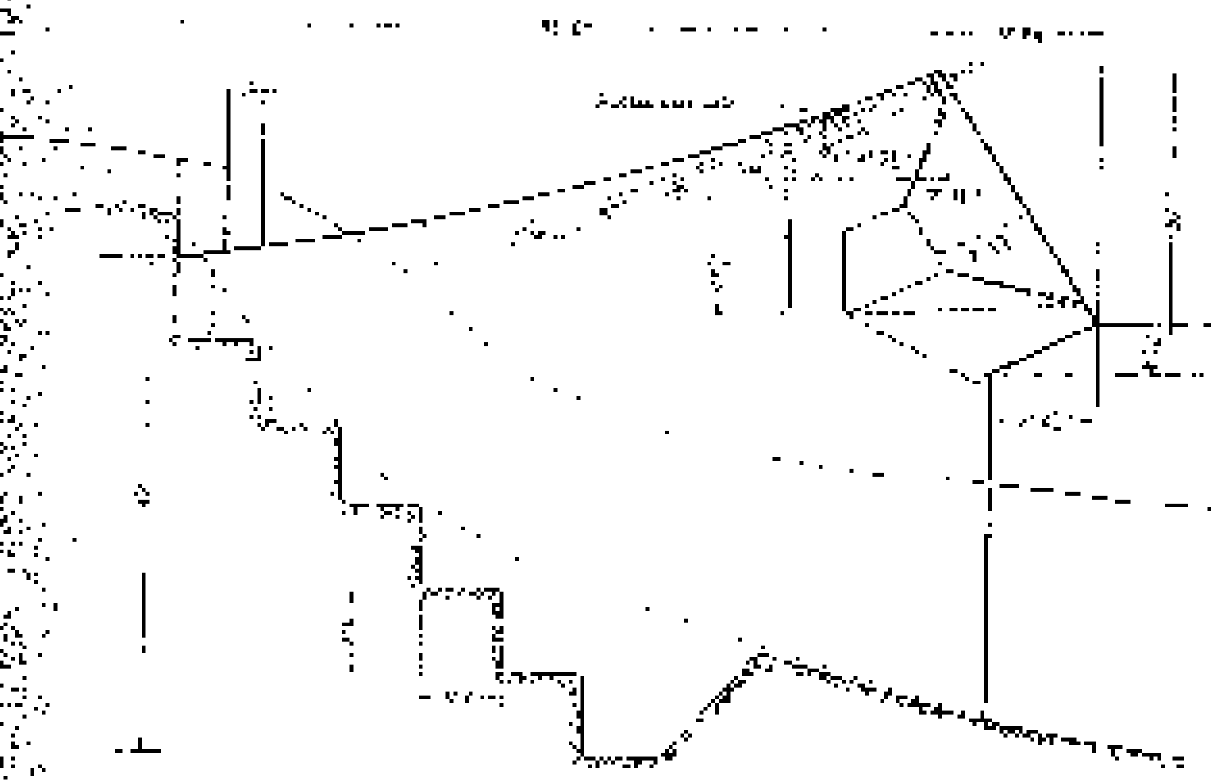


FIG. 14. DETAIL OF ARCH ABUTMENT

from the portion of the bridge between the expansion joints are carried back to the crown of the arch as a fixed point, the actual points of anchoring being 150 ft from the arch centreline where local 'abutments' were formed.

24. The diaphragms are spaced at approximately 50-ft centres. It is necessary to place a diaphragm under each pier to transmit the load from the deck equally to each rib. As the effect of all lateral forces on the deck is transmitted to the arch ribs through these diaphragms in the form of two moments (from the base of each column) and a large couple, it is necessary to have at least one intermediate diaphragm between adjacent piers to prevent a cumulative distortion of the arch ribs. Consequently each rib is a voussoir arch consisting 108 box-section units and 19 solid 2-ft thick diaphragm units. The 3 in. wide joints between the units were filled with poured in situ concrete.

Design

25. The problems involved in setting out the arch rib units were, to a certain extent, affected by the design, the appearance and the construction of the bridge. In the first place it was decided that because the method of construction dictated that all the arch ribs would stand unloaded except for their own weight for some time after de-centering, and because in any case the stresses caused by their self-weight were a large proportion of the final total stresses, the arch would be made perfectly funicular under its own weight. Secondly, it was realized that the 3 in. wide joints would be plainly visible, and that therefore they should be disposed in an acceptable pattern. Lastly, in order to keep the number of units to be erected to a minimum and use the lifting equipment in the most efficient way possible, all units should be as near as possible to 50 ton in weight. To fulfil these requirements three inter-connected computer programmes were written specially for the analysis of the arch.

26. To set out a structure of this size and shape with sufficient accuracy, both intrados and extrados must be related in some way to geometric curves. Preliminary calculations and a certain amount of trial and error had produced geometric curves which gave approximately the right ratios of stiffness at the critical sections. The first programme was written to calculate these curves for given depths of the arch at springing and crown and thence to calculate the profile of the centroid of the section. It then produced the area, moment of inertia, section modulus, etc. at a large number of sections across the arch.

27. This information was fed into the second programme which calculated the influence lines for unit loads at these sections, and also the moment and strain due to unit thrust and the moment and thrust due to unit strain. This programme also calculated the funicular profile of the arch under its own weight. This could then be compared with the centroid profile obtained from the first programme. When the calculated stresses were acceptable and at the same time the centroid and funicular profiles were in fairly close agreement, a satisfactory shape had been obtained for the arch ribs. In fact, the geometric curves finally used for setting out the arch resulted in a centreline which did not deviate by more than ½ in. from the funicular profile over its entire length.

28. The third programme calculated the dimensions of each precast voussoir unit and the co-ordinates of each corner of the unit in its final position

of the arch. The information supplied as input data, apart from the equations to the intrados and extrados curves, consisted solely of the horizontal distances to the front and back upper corners of the unit from the centreline of the arch. These distances were determined on the basis that each unit should be as nearly as possible 50 ton in weight and that the widths of adjacent units should not be noticeably different. As the positions of the diaphragms under the columns supporting the deck were fixed and the number of units between each pair of adjacent piers determined by the weight limit, some averaging out of the distances between the joints was necessary to avoid any sudden or irregular change in the size of the units.

29. Because it was essential that all the joints should be normal to the longitudinal axis of the arch, the diaphragms could not be placed vertically and as a result the design of those diaphragms which carry supporting columns was complicated by the angle between the diaphragm and the column. The solution was found by casting a wedge-shaped rib on to the back of each diaphragm unit. This rib fitted inside the adjacent box unit and was used as the end-block against which the vertical prestressing bars in the columns were stressed (Fig. 15).

30. The corresponding diaphragm units in the four ribs were made into a single unit which effectively distributes the loads from the columns equally to each rib. The 9 in. wide joints between the diaphragm units were cast in situ and the whole unit transversely prestressed by twelve ¼-in. dia. strand cables. These tendons are rather short for this particular type of cable but a high concentration of load was required because of the restricted area at the anchorages; in addition, two different systems of prestressing were already being employed and Macalloy bars could not be used because of the curvature required to pass the tendons round the apertures in the diaphragms. These apertures had to be fairly large, not for reasons of access, but in order to reduce the weight of the larger units to 50 ton.

31. Since, for the reason given later, the top flange of the arch is also made continuous by in situ joints, each diaphragm is in effect a T-section with a top flange some 30 ft in width. Assessing the stresses in the extreme fibres of this section from both the external loading, and more important, the applied prestress, was therefore a difficult problem. As there is no continuous transverse

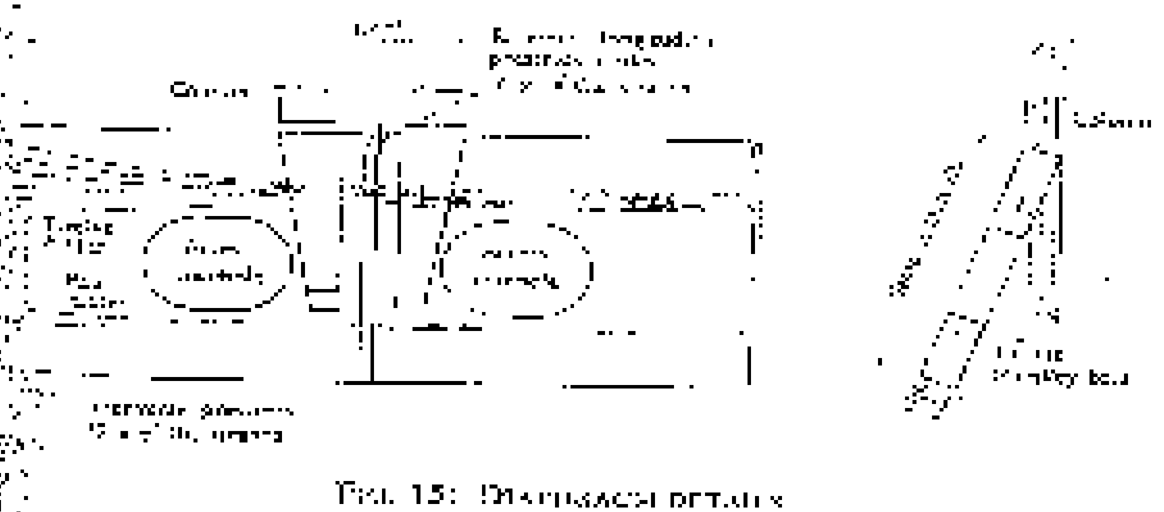


FIG. 15. DIAPHRAGM DETAILS

reinforcement in the top flange, nor any reinforcement projecting into the in situ joints between the ribs, it was considered essential, in order to maintain monolithic action of this top flange, that there should be a residual transverse prestress of at least 150 lb/sq. in. across the flange joint at all times. An analysis based upon a modification of the theory of a beam on an elastic foundation indicated that the minimum transverse prestress would be about 89% of the average in the outer joints and about 99% of the average in the centre joint.

32. The primary analysis of the arch was therefore carried out on an elastic basis. It was necessary, however, to evaluate the factor of safety against failure by overloading and through instability of the arch.

33. The relative magnitude of the bending moments and thrust is such that the live-load can be multiplied many times before the theoretical ultimate state is approached at any section and, as the self weight of the arch itself is theoretically funicular, increasing the dead load as well only serves to increase the factor of safety. It appeared illogical to increase the superimposed dead-load but not the self weight of the arch, so a realistic factor of safety against overloading was not readily obtainable.

34. Two possible cases of instability had to be considered: vertical instability of the whole arch and horizontal instability of a single rib during erection. Investigation of the vertical stability showed that under full dead load conditions where H is the horizontal force and Hcr is the critical value:

$$\frac{Hcr}{H} = E \cdot \frac{1 \cdot 43}{10^6} \quad (E \text{ in lb/sq. in.})$$

35. The horizontal stability of a single rib was such that under the action of the self-weight of the rib alone:

$$\frac{Hcr}{H} = E \cdot \frac{1 \cdot 775}{10^6}$$

36. As the average age of the units in a rib at the time of de-centering was likely to be at least three months it was felt that the effective value of E was not likely to fall below $2 \cdot 0 \times 10^6$ indicating a factor of safety against instability exceeding 2·5.

37. The Department's Consultants, Société Technique pour l'Utilisation de la Précontrainte (S.T.U.P.), however, regarded these values as optimistic and were of the opinion that the factor of safety against vertical instability should be at least 2·5 assuming an effective modulus of $1 \cdot 4 \times 10^6$ lb/sq. in. The effect of the adoption of this factor of safety and modulus was to increase the depth of the arch at the crown from the 12 ft originally intended to 14 ft. This increase in dimensions led to practically no change in the elastic stresses as the reduction in the loading stresses due to the increased cross section was cancelled out by the increase in stresses due to temperature resulting from the greater stiffness of the central part of the arch.

38. The Department's Consultants also felt that the continuity of the diaphragms was not sufficient in itself to guarantee homogeneous action of the four ribs in resisting horizontal instability as the diaphragms would be subject to very high shear forces across their weakest plane. The decision was therefore taken to make the top flanges of the ribs continuous. This also considerably reduced the stresses caused by wind loads and conveniently allowed

another safety measure to be incorporated. The Department's Consultants, while agreeing that the load factor in the normal sense was not readily assessable, pointed out that failure must be preceded by the formation of a mechanism having four hinges, two in one direction and two in the other. If the ultimate moment in one direction were to be increased by the inclusion of some tensile content in these in situ strips in the top flange, the theoretical load factor must necessarily be increased. Accordingly each in situ strip contains fourteen, seven ½-in. dia. strand cables. These were used because they represent a concentrated tensile force, they could be obtained in the necessary continuous length, and they could be partially stressed to compensate for the considerable later shortening of the arch. The reinforcement provided actually amounts to less than 0·14% of the cross-sectional area of the concrete.

19. Each arch rib was sprung by jacking approximately 3½ in. at each quarter-point. For this purpose, the diaphragms at these points were made in halves which were separated by inflating a ring of Freyssi flat jacks placed around the perimeter to correspond with the section of the rib. In this way, not only was the centering freed over most of its length, but also as the quarter-points coincided with the points of contraflexure of the rib under linear strain, the immediate and future shortening of the arch was fully and exactly compensated. Thus the future shortening due to creep and shrinkage would only result in a reduction of the bending stresses imposed by the jacking, and much of this reduction would have taken place before the stresses due to the weight of the superstructure and live-load were applied. Each ring of jacks contains forty 14·8 in. dia. jacks, corresponding to the 15 in. thick top and bottom flanges, and twenty-six 11·8 in. dia. jacks in line with the 12 in. thick webs of the arch rib. It was originally intended to have three such rings of jacks at each jacking point. On the advice of the Department's Consultants this was increased to four rings of jacks on each side in order to give a greater margin to meet possible contingencies. However, it did not prove necessary to use the additional capacity as the required extension was achieved, in each instance, with three ring jacks.

20. A group of three jacks at each end of each vertical row was isolated from the main circuit so that they could be used to adjust the centre of pressure within the arch. These jacks were operated from a separate pump and the pressure in each group of 'trimmer' jacks could be adjusted by controlling the flow by means of needle-valves. The trimmer jacks were operated at a constant pressure of 1200 lb/sq. in. except when trimming action was required. A difference of ±100 lb/sq. in. in the pressures in the trimmer jacks could provide an eccentricity of about ½ in. vertically or 1 in. horizontally, equivalent to maximum bending stresses of ±10 lb/sq. in. and ±20 lb/sq. in. respectively. It was envisaged that the main function of the trimmer jacks would be to correct distortions of the arch rib caused by unavoidable temperature gradients across the rib. In this respect a differential pressure of ±150 lb/sq. in. produced the same distortion as a linear temperature gradient of 1°F through the depth of the rib, while a differential of ±95 lb/sq. in. was equivalent to a 1°F gradient across the breadth of a rib.

21. Measurement of pressures, deflexions, and rotations was deemed necessary to ascertain the behaviour of each arch rib during jacking and to

determine the control required. Readings were taken at 32 positions on each arch rib throughout the jacking operation. The positions at which readings were taken were as follows:

(a) at each of the two jacking points: two pressure gauge readings in the main jack circuit, one pressure gauge reading in each trimmer jack circuit, one dial gauge indicating jack displacement at each corner of the arch rib, making ten readings at each point—twenty in all.

(b) at each quarter point and the crown: one measurement of vertical deflexion, one measurement of horizontal deflexion, one measurement of (flexural) rotation in elevation and one measurement of (torsional) rotation, making four readings at each point, or twelve readings in all.

In addition a measurement of the movement of the crown of the arch along the length of the bridge was made.

42. Observations through fixed instruments on the ground to scales attached to the arch were made to measure horizontal linear movements. The vertical movements were calculated from readings taken by theodolites onto targets attached to the arch. Rotations were measured by Talyvel electronic levels.

43. The averages and the differences of the above readings, where appropriate, enabled the five controlling functions to be continuously checked. These were the main jack pressure, the trimmer jack pressure differentials, the jack displacements, and the profiles of the arch in plan and elevation. The rotations of the rib itself and the angular changes across the jacking points were used only as a check against local misbehaviour of the arch. Any significant deviation from the ideal profiles was corrected by adjusting the trimmer jack pressures.

44. The relationships between trimmer jack pressure differentials and the resulting deflexions and rotations were drawn up in chart form so that the necessary corrections could be applied without recourse to calculation. Had the capacity of the trimmer jacks to apply compensating deflexions been inadequate it would have been possible to develop further correcting moments by "shunting" main jacks out of the circuit, but this would have resulted in greater pressures in the remaining jacks and it was decided at the outset not to resort to this measure unless absolutely essential.

45. The greatest deviations from the theoretically correct profiles were 1 in. in elevation and 2 in. in plan, although, as the latter included the effects of the rotation of the abutments under eccentric loading, the real deviation from the straight line was less than ½ in.

46. Progressive readings of the main jack pressure and jack displacements were used to investigate the elastic and early plastic behaviour of the arch and hence to confirm the calculations of the probable final settlement.

47. The jack displacement corresponding to the vertical profile at any stage had been calculated; it followed that any jack displacement in excess of this was equivalent to the shortening of the rib which had been induced. In this way the unknown reaction between rib and centering was eliminated from the calculations and it was possible to calculate the effective modulus of elasticity at every stage of the jacking. The strain did not all take place at constant stress but, as more than two thirds of the final thrust was applied by inflating the first row of jacks in a few hours, the effect was not significant.

48. Two other unknown factors, the spread of the abutments and the compression or tension actually present in the rib before jacking commenced due to shrinkage and changes in climatic conditions, could be fairly accurately eliminated (but not separated) by extrapolation from the curve of jack displacement plotted against thrust or jack pressure.

49. Readings on all four ribs were very similar and adequately confirmed the assumption made in calculating the amount of jacking required. The initial elastic modulus was approximately 6×10^6 lb/sq. in. and the readings indicate that the probable ultimate modulus will slightly exceed 3×10^6 lb/sq. in. This value includes the effects of both shrinkage and creep and corresponds to a total deferred strain after jacking of 450×10^{-6}.

50. The calculation of the necessary jack throw was complicated by the need to have all four ribs approximately at the same level at the time when transverse joining and stressing of the diaphragms was due to take place. Examination of the construction programme enabled an allowance for this to be assessed but it was unavoidable that the final levels which the ribs would have reached individually are different from those imposed upon them by transverse stressing before their differences in age had become insignificant from the point of view of creep and shrinkage. The effect on the cross fall of the bridge induced by shrinkage and creep after transverse stressing will not, however, exceed 1 in. in the width of the bridge and the effects upon stress are equally insignificant.

Construction

51. The original tender submitted in October 1957 was for the construction of a concrete arch bridge 910 ft in span with 91-ft span approaches. It proposed a method of construction which envisaged the prefabrication of the central 500 ft of each of the four ribs upon prepared falsework built on a pontoon. On completion each piece was to be lowered onto the outer quarters of the ribs previously cast in situ on fixed falsework. The lowering was to be done tidally and by controlled flooding of the pontoon. This method of construction was adopted to meet the requirements of the design specification which imposed an upper limit on the crown roadway level and a navigation clearance required during construction. Between these limits there was insufficient depth to accommodate both permanent works and fixed falsework.

52. The design and the construction method were accepted but subsequently the navigation clearance required during construction was reduced and a system of falsework founded upon piles with a deep truss over the navigation channel became acceptable. Accordingly a scheme was proposed and accepted using continuous centering to support precast voussoir units.

53. Preliminary schemes for the falsework and cofferdams were prepared and at this time it was realized that the rock under the Gladesville abutment was unsatisfactory. The abutment was to be constructed in a large circular double-walled cofferdam and any fissured rock uncovered behind the abutment would involve an extremely costly enlargement to the cofferdam. It was therefore proposed that the length of the arch be increased to 1000 ft with the further advantage that the Gladesville cofferdam would now be three-sided only and any unsatisfactory material behind the abutment would be removed by excavating back into the foreshore.

54. Once this major change had been accepted, the Contract had assumed its final shape. Despite changes to the original drawings which included widening of the Gladesville approaches and the modifications made on the recommendation of S.T.U.P., no significant alteration to the construction procedure was necessary.

55. The work can be conveniently divided into two periods, the first being the preparatory or manufacturing period, and the second the erection period. During the first period the work can be divided into six construction areas:

Drummoyne abutments and approaches
Gladesville abutments and approaches
Falsework, pile driving, falsework erection
Woolwich Casting Yard
Drummoyne Beam Casting Yard
Gladesville Beam Casting Yard

The second period has three major divisions:

Launching of deck beams and deck construction
Erection, jointing, and jacking of arch
Completion and erection of spandrel columns.

Drummoyne abutments and approaches

56. The Drummoyne Bank was used throughout as the main site and a small wharf was constructed on the downstream side of the bridge, with about 10 ft of water at low tide. A 10-ton Henderson derrick with 140-ft jib was located to cover this wharf, the cofferdam, and the arch abutment in this area. It also had sufficient height to construct the expansion joint pier—Pier 4 on the main abutment.

57. All excavation was carried out by A. Bradshaw (Excavations) Pty Ltd. Construction of the deck abutment and the approach piers was with a 22 RB crawler crane. All concrete was supplied by mixer trucks and placed with side dumping skips specially designed for stiff concrete. Plywood shutters were used and the pour size did not exceed 6 ft 9 in. for the columns.

58. The prestressing tendons in the columns consisted of 1⅛-in. Macalloy bars. They were wrapped with a double skin of Denso tape, the first skin being a petroleum-based anti corrosive, the second skin a bitumen-impregnated tape applied with heat. No major difficulties were experienced in stressing which was carried out with a Macalloy jack and a Tangye hand pump.

59. The face of the Drummoyne arch abutment is about 25 ft out into the river from the high water mark. It was decided that the most satisfactory method of carrying out the excavation and concreting was in a sheet pile cofferdam enclosing the whole abutment. As it was desirable to leave as much working space as possible, a cofferdam in the form of an arch was designed with two main support ribs bearing on concrete walls projecting from the steep rock shore. The two steel ribs were prefabricated and assembled on timber falsework. B.H.P. 30 lb/ft sheet piles were then driven using the ribs as formers.

60. The two concrete abutments performed a double function: they provided a sealing wall to close the cofferdam off against the cliff face and they transferred the loads from the two ribs back to the rock. These two abutments were built by driving timber sheeters down to rock, dividing the

material out of the bottom, and tremmying concrete onto the bare rock. After the cofferdam had been dewatered two blows occurred—one under each abutment. These were caused by the presence of pockets of very stiff clay which were left by the airlift. Both the cavities caused by the blows were successfully repaired with tremmied concrete. After the concrete and the pile driving had been completed, some pressure grouting was carried out in the concrete abutments and in the rock face adjacent to them.

61. The cofferdam proved successful and was extremely watertight. It was dewatered by a 6 in. centrifugal pump with an automatic level switch. Excavation was carried out with care as some concern was felt over the effect of blasting on the stability of the cofferdam. Trenches 4 ft wide were removed by jack pick to isolate the central body of the rock from the cofferdam walls before blasting was commenced. Concreting was by means of mixer trucks discharging into a 10-cu. yd holding hopper. Concrete was then distributed from the hopper by 2-cu. yd roller-bottom buckets on the 50-ton derrick. The concrete pours were broken up to allow for shrinkage, the largest pour being approximately 400 cu. yd. Shuttering consisted of plywood panels and the concrete was vibrated with 4-in. immersion vibrators.

Gladesville abutments and approaches

62. The Gladesville bank was shallow, shelving and rocky. A 10-ton guyed derrick with 110-ft jib was used to cover the abutment, but it was not possible to arrange it to cover the whole of the cofferdam and sufficient of the water to be of use for loading barges. Construction of the approaches was identically as for Drummoyne. However, the extra length of the crosshead in the Gladesville approaches involved the use of multiple crane hire for the erection of crosshead reinforcement cages.

63. The face of the Gladesville abutment was approximately 80 ft into the river. Bed rock was exposed in the area with some small pockets of silt. The depth of water at the abutment face was approximately 5 ft at mean tide. The original intention was to form an earth dam around the excavation area with an impervious clay core. Observations of the current and contours in the area where the toe of this earth dam would be led to the conclusion that the rate of erosion of material from the toe of the bank would make an earth dam unsatisfactory. It was therefore decided to use a timber sheet cofferdam. Timber piles were socketed into the rock and propped to form king posts which supported the walings and sheeters. A tremmied concrete bottom was formed at the toe of the sheeters to give anchorage and sealing onto the rock. Some concern was felt at one stage owing to the activities of *Teredo* worm in the timber sheeters and a second skin of timbers was added to the inside of the cofferdam.

64. Excavation and concreting were carried out as for Drummoyne, except that owing to the lower foreshore it was possible to remove a large part of the spoil by truck instead of all by crane as at Drummoyne.

Falsework

65. The main condition imposed on the use of falsework was that a navigation opening of approximately 200 ft be provided for the use of river

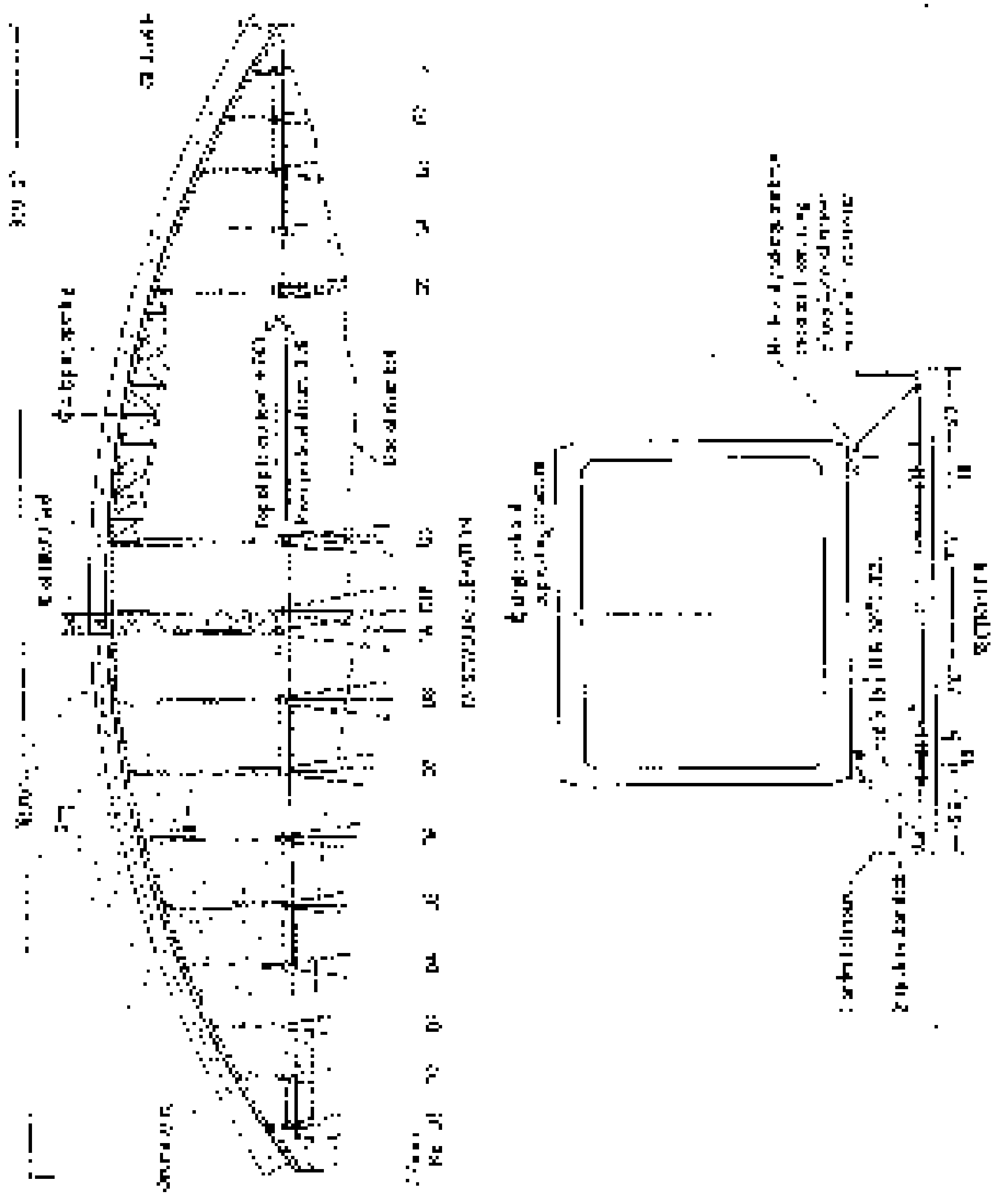


FIG. 10.—DETAILS OF FALSEWORK

65. It was originally intended to span the centre of the stream with a 220-ft truss and to provide two sets of universal beams or small trusses for the sides. Two hoists were to be provided at each end of the truss. In this way work could proceed continuously in placing and jointing of units in one rib while jointing on the previous rib was proceeding. This scheme was prepared in outline and submitted to the Maritime Services Board. The Board made the following conditions:

(a) 200-ft clear opening was required,
(b) centre of navigation opening to be as near Gladesville as possible with a minimum of 160 ft from centre of arch, and
(c) a minimum clearance of 80 ft above mean high water springs.

These requirements fixed the position and shape of the truss and enforced the use of a single central hoist. They also meant the rejection of the use of double ... of minor falsework (Fig. 16).

67. For the foundations of the falsework various possibilities were investigated including large diameter tubes and timber piles. It was finally decided that 19-in. dia. steel tubes ¼ in. thick presented the most economical solution. This diameter pipe is formed by the rolling of a 5-ft steel sheet with a spiral seam. The thickness of the plate gives a reasonably long life without excessive weight. Nevertheless, owing to some concern during the early part of the work a cathodic protection scheme was installed. Altogether approximately 30 000 ft of steel tubes were used. A welding bench was set up under the Henderson derrick. On this the pipes were rotated by a small electric motor as welding of the butt joints was carried out.

68. Owing to the high wind-loading on the falsework when carrying a complete rib the great majority of the bents had piles raked in four directions. This precluded the use of a normal pile frame. John Jamieson Construction Pty Ltd, who were sub-contractors for the pile driving, designed a rig with swinging leaders and a 50-cwt drop hammer. This arrangement was based on the probes they had carried out when the possibility of falsework was first discussed. When the first four bents had been driven it became apparent that the rock was lower than had been thought and the 50-cwt monkey was incapable of driving the steel pipes down to the rock through a stiff bed of clay. A pile test was carried out which showed excessive settlement under loads of 50–60 ton.

69. A new rig was designed and constructed with a 5-ton drop hammer working in 80-ft leaders. With this rig it was found possible to drive piles to rock through 90 ft of overburden. It was also possible to drive a raking pile in any direction without moving the aspect of the punt. In the main navigation pier support at pier 10-11 the congestion due to the number of piles and the displacement of the toes due to the raking was so severe that a model of the whole of the bent was made to facilitate the pitching of each pile (Fig. 17).

70. Pile driving was possible only on the Drummoyne side of the river. On the Gladesville bank the overburden nowhere exceeded 6 ft in depth and was usually less. These piles were potted into rock. It was impracticable to drill a fitting hole, so the piles were founded in an oversize hole, which was then filled with tremied concrete. Shear connectors were welded to the outside of the pile and rock holes where uplift was expected. Setting out of the piles presented a major problem as the contours of the river bed had first

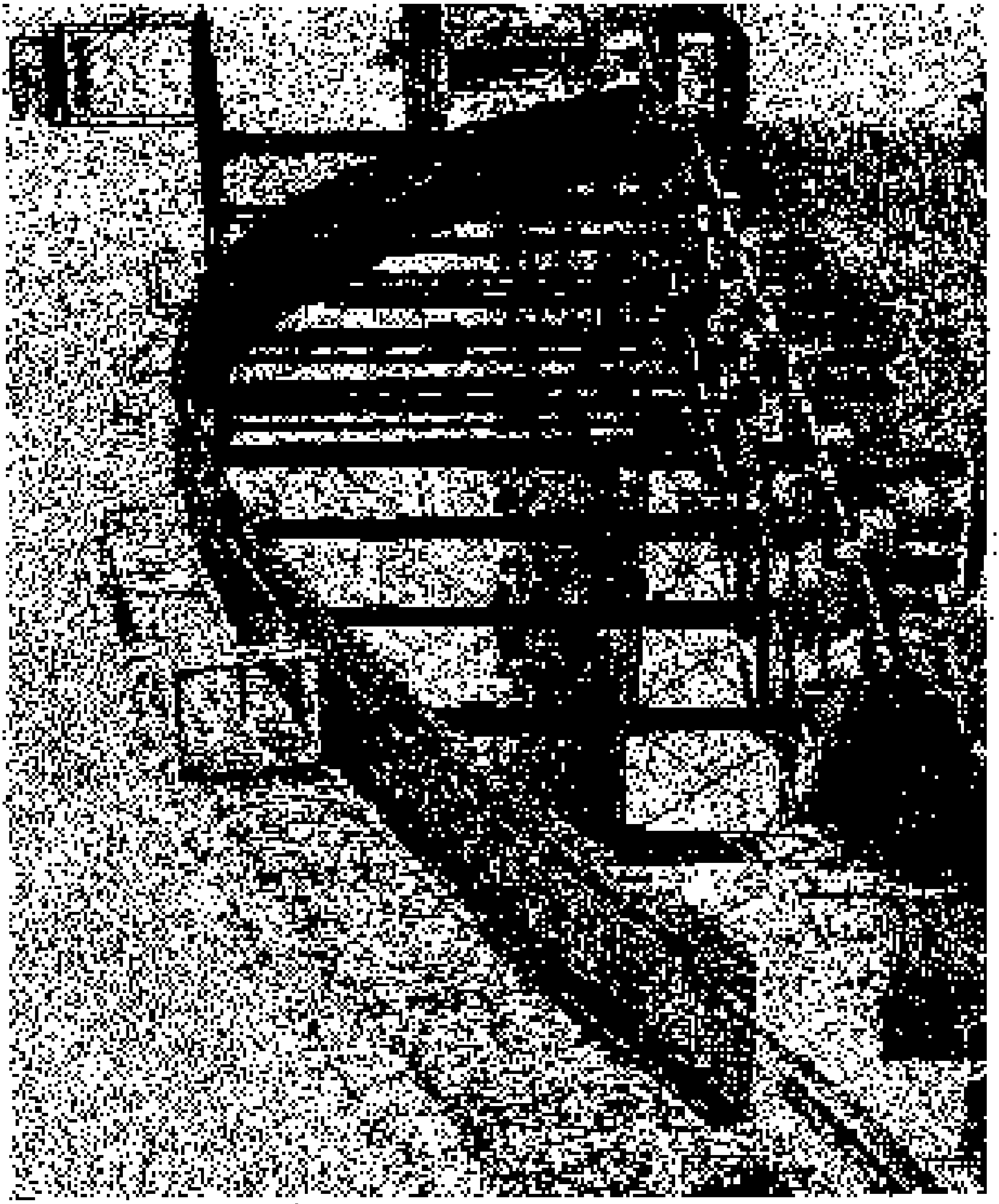


FIG. 17: VIEW SHOWING LAYOUT OF PILE CAPS

to be plotted before the root hole positions could be set out. Final setting out was carried out by sextant from a number of prepared marks in the area.

71. After the drilling rig had completed work on a bent, the holes were inspected and cleaned by a diver. Those requiring rock bolt anchorages were drilled, rock bolts fixed, and tested by a floating crane. The piles were pitched into the holes by the diver and guyed into their correct position. Owing to the rugged nature of the river bed it was found that interference took place between some of the piles and cliff edges. These were trimmed to allow the piles to take up their correct position.

72. Pile caps were reinforced concrete with steel connecting beams. Originally reinforced-concrete beams were suggested but RSJs were thought to be easier to demolish. Casting was carried out at first by placing two [illegible] concrete agitators on a punt, filling them from the bank and discharging into the forms. This did not prove satisfactory and the later part of the work was carried out by a 22 RB on a punt and side discharging skips brought out on a tug.

73. The steel tubes for the falsework columns were rolled by Humes Ltd. of Melbourne. They were then fabricated into their correct lengths, the caps and base plates fixed and cross bracing welded into place by the sub-contractor at a temporary shop at Woolwich Dock two miles from the bridge site. The columns were then lifted into the water in pairs and floated up the river to be stored awaiting erection. The shorter columns on both banks were fabricated and erected at the main site. The columns weighed about 30 ton per pair except for those supporting the navigation truss which weighed 32 ton and 40 ton.

74. Universal beams were used to support the rib units and transfer their weight to the falsework columns. The beams were of high tensile steel (B.S. 548) 36 in. × 16½ in. × 260 lb/ft and ranged in length from 45 to 60 ft. They were rolled by Dorman Long & Co. Ltd at their newly-opened universal mill at Lackenby, specially for this project, being the first beams of this size rolled in steel of this quality. When the beams were being prepared evidence of severe lamination was found in one 50-ft beam and all other beams were checked ultrasonically for similar defects. None was found. No suitable replacement was available locally so a mild steel universal beam was used and plated to increase its section properties. The universal beams were received [illegible] by Henderson derrick, each bay was prefabricated with 8-in. × 6-in. runway beams, cross bracing, handrails, and scaffold planks. Six bays were stacked one above the other on the Drummoyne wharf. The weight of a complete standard bay 60 ft long was approximately 26 ton.

75. The navigation trusses were approximately 220 ft long with a maximum depth of 23 ft and a weight of 92 ton. They were fabricated by the Sydney Steel Company at their works in Marrickville. Before sending them to the site they were preassembled flat as it was impossible to erect them in their true position in the shop. It had been stipulated that no piece was to exceed 5 ton in weight. The pieces were brought on the site and assembled on the Drummoyne pile caps by a 10-ton floating steam derrick. The bottom chord was laid horizontally on the caps and a hinging point arranged to allow tilting to the true position when lifted. Trimming to level was by water ballast.

76. Erection of all the main falsework was carried out by the floating crane

Titan, belonging to Cockatoo Dock & Engineering Co. Pty. Ltd. It has a capacity of 150 ton at 90-ft radius with a maximum height of hook of 154 ft. The first three columns and spans on each side of the river were erected by the shore cranes. The procedure with *Titan* was then as follows.

(*a*) 1st visit *Titan*—1 week:

Six prefabricated spans were placed on punts. Five pairs of columns were erected at the central tower. Four pairs of columns and four spans were erected on the Drummoyne side, one pair of columns and one span on the Gladesville side (Fig. 18).

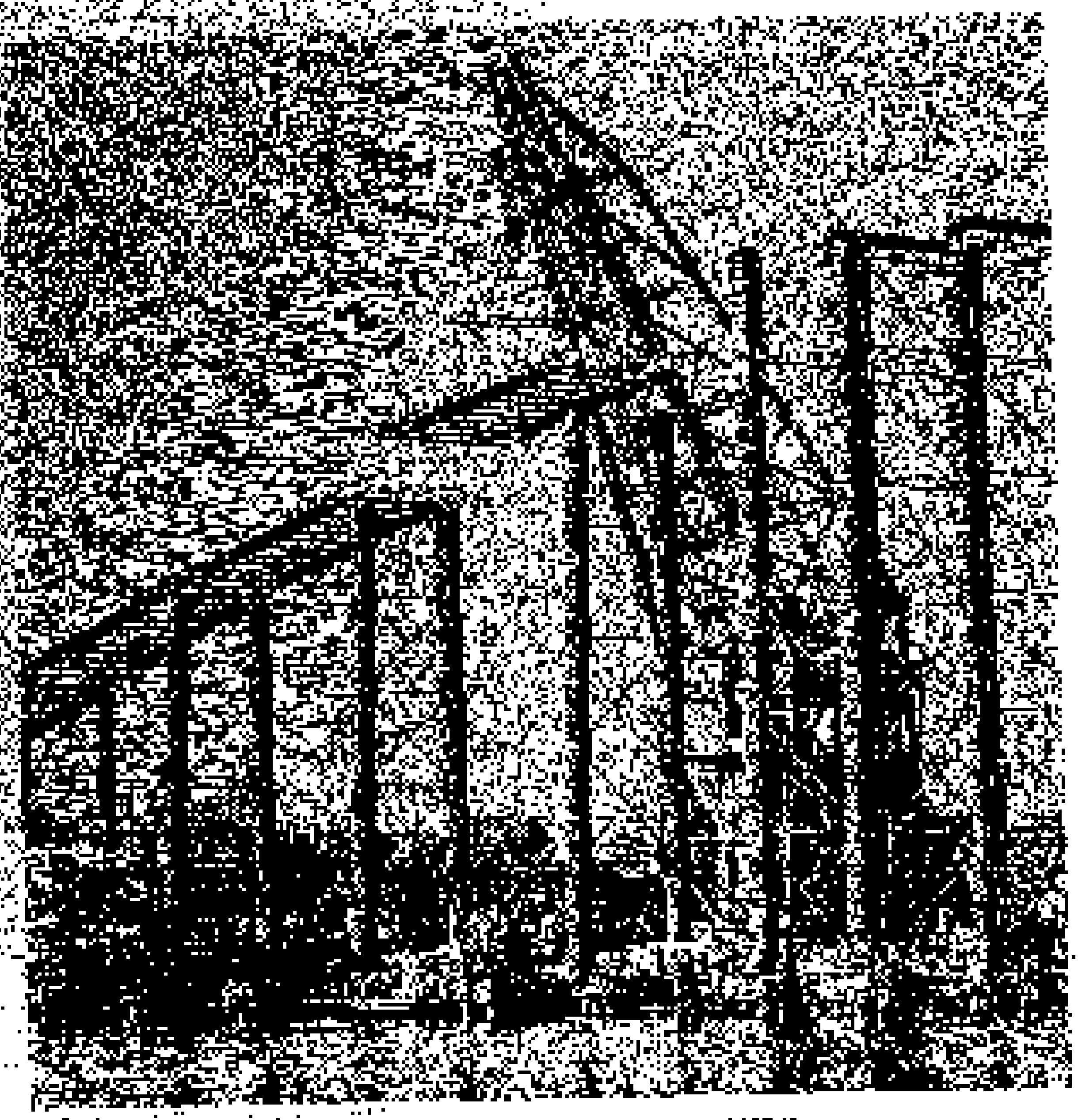


FIG. 18. TITAN ERECTING FALSEWORK

(b) 2nd visit—5 weeks later; 3 days' work:

One pair standard columns and both navigation truss support columns erected, all remaining universal beam spans placed, 50-ton hoist placed at low level. It was intended to erect the trusses but two days of gale force winds intervened.

(c) 3rd visit—2 weeks later; 2 days' work:

Navigation trusses erected.

In these three visits, totalling eleven days, 800 tons of steel were erected.

77. The plant and machinery incorporated in the falsework included the main 50-ton hoist, the transporter trolley, downslope trolleys, 5-ton and 14-ton winches.

78. The main 50-ton hoist was designed as an 8-wire hoist to set down 50-ton ribs; an 80-hp electric motor driving four hoisting drums through three helical gear boxes and four pinion shafts, arranged to operate in pairs. On each pair of drums a 1-in. wire rope passed from the first drum down around a sheave on the lifting beam up over an equalizing sheave at the winch, down to a second sheave on the lifting beam and back to the second drum of the pair. Hoisting speed was about 12 ft/min. The erection of this hoist presented a special problem in that it was approximately 170 ft above the water, beyond the reach of any available crane. The problem was overcome by erecting the hoist at arch spring level about 140 ft above the water, jacking it up on its own stanchions and inserting the cross bracing as the hoist was lifted.

79. The transporter trolley was passed under the rib unit after it had been lifted by the hoist. A fixed winch on the upstream side of the central tower pulled the loaded trolley to the rib being constructed. The empty trolley was pulled back to the loading position by counterweight. The control of the transporter winch was in the main hoist control cabin.

80. Two downslope trolleys were provided, one for each side of the rib. They were equipped with jacks to lift the unit and with traversing jacks to move it sideways.

81. One 5-ton winch mounted at pile cap level was provided to control each of the two downslope trolleys. Each winch was driven by a 15-hp motor with rope speeds of 20 ft/min in low gear and 40 ft/min in high gear. The winch drum was arranged with two wires operating in opposite directions. One wire passed over a sheave to the tow bar of the trolley, the other over a similar sheave, down the arch, round a return pulley, and back to the tow bar, the two ends of the rope being attached to the bar with a single shackle. The use of the long tow bar enabled the winch driver to drive the trolley on to the transporter without interference from wire ropes. No ropes led across the central tower. These winches would not have been adequate to restrain the loaded downslope trolleys on the steepest parts of the arch and their use was limited to 60 ft on either side of the centre where it was not possible to use the more robust main lowering gear owing to the size of the wires, trolley and the height of the units themselves.

82. At the steepest part of the arch the 50-ton rib units required a maximum restraint of approximately 26 ton but the difficulties of manufacturing a 26-ton winch were such that an alternative means of lowering had to be found. The ribs were therefore lowered down the falsework by 14-ton winches of a special design. Three drums were mounted in line, all driven by a 30-hp motor to

give a rope speed of 20 ft/min. The two lowering ropes were wound on the outer drums. From a central drum a wire rope was wound in the opposite direction, led through a four-part tackle to a 20-ton counterweight suspended from the falsework. The operation of the counterweight ensured that the winch never had to deal with a resultant load greater than 14 ton.

83. To ensure that the heavy 1½-in. ropes were kept taut at all times, back tackles of ½-in. wire rope were fixed to the counterweight frame passing down the falsework and back to the main hoisting eyes. These automatically took up all slack and kept the main hoisting ropes at constant tension when not in use. The main hoisting wires were connected through an equalizing bar

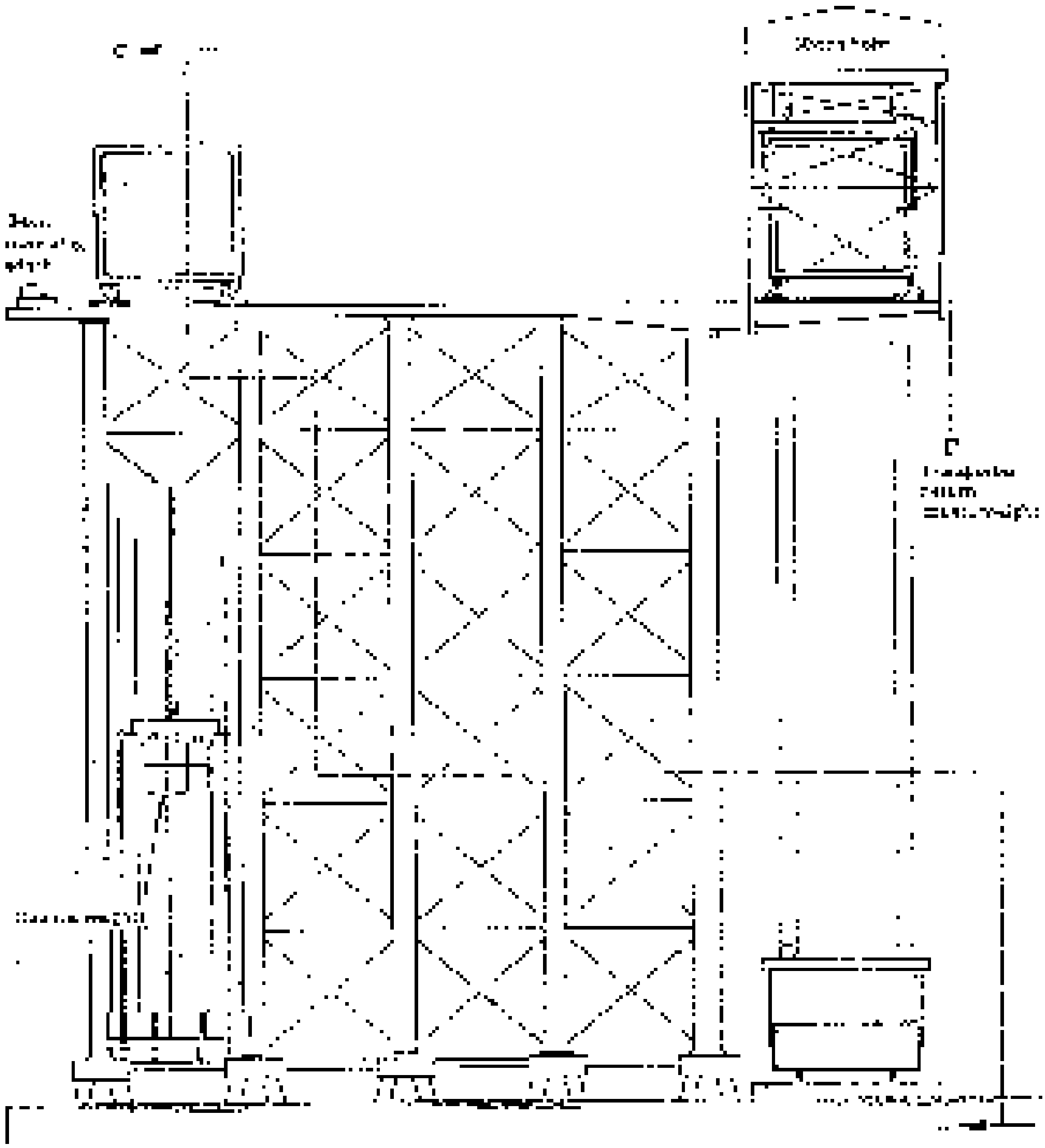


FIG. 23(a): LAYOUT OF UNITS: ELEVATION

its pivoting point very close to the centre of gravity of the unit so that there was no resultant overturning moment on the trolley, nor, despite the steep slope, was it necessary to secure the unit to the trolley. Owing to the very high loading when most of the units for a rib had been placed, it was necessary to remove the 50-ton counterweight while placing units in the central 120 ft of the arch by means of the lighter gear (Figs 19(a) and 19(b)).

Arch rib unit casting yard at Woolwich

184. Lack of space at the bridge site necessitated casting elsewhere and a site at Woolwich was chosen. Transport was by water.

185. The layout was planned with sufficient area to cast a complete rib without moving any units to storage. Low level trolleys travelling on tracks passing through the casting beds were used to move the units which had to be cast in order of placing. The programme was planned on the basis of casting two units per day using five complete sets of shutters. The units were cast on end and their external dimensions varied from 20 ft × 12 ft 7½ in. to 20 ft × 14 ft, the height as cast varying from 7 ft 7 in. to 10 ft 0½ in.

186. Owing to the order of casting it was found necessary to ensure that each shutter would be capable of casting any size unit. The shutters were therefore

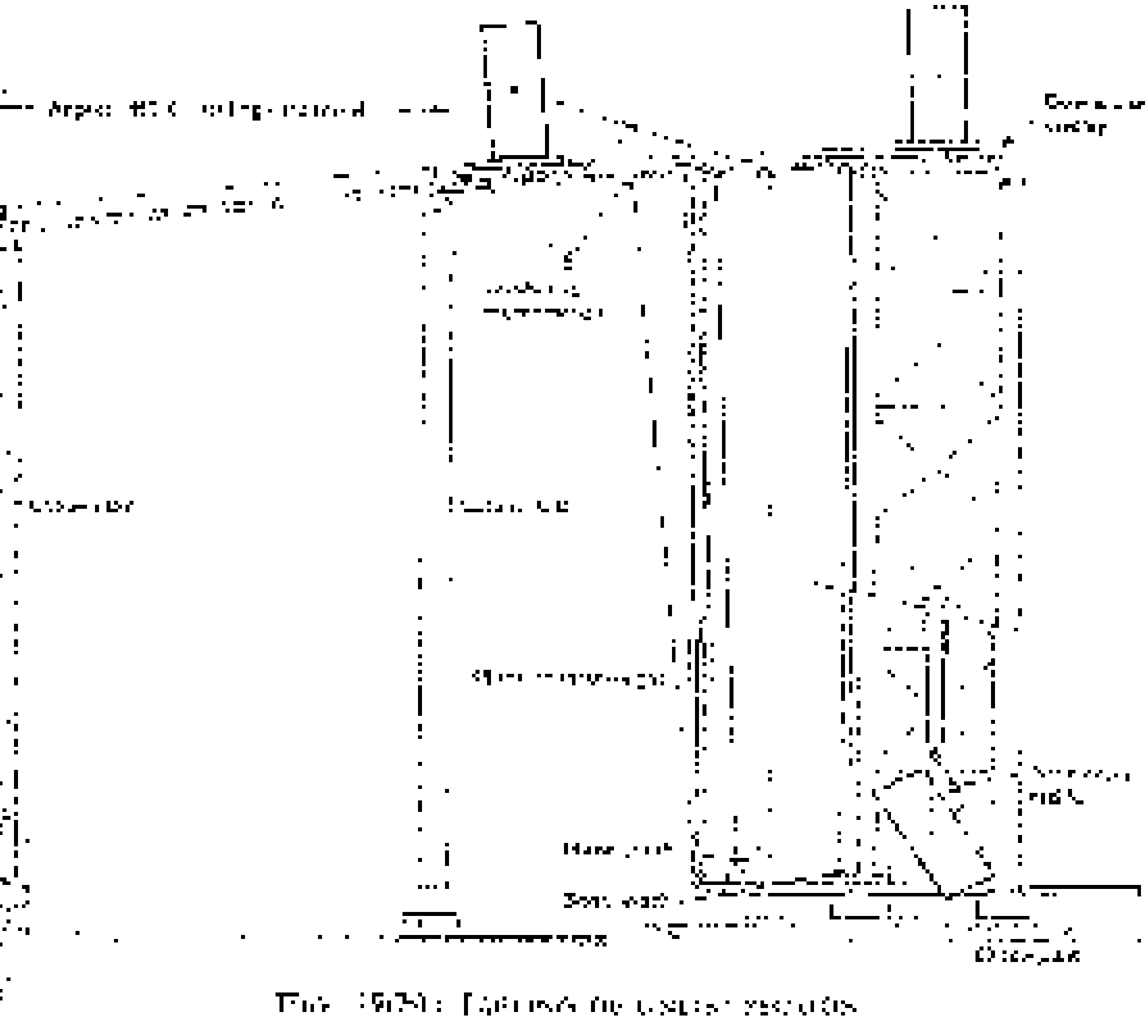


Fig. 19(b): [illegible]

designed to cover all the variations listed above. Steel shutters were used throughout, supplied by Rapid Metal Developments (Australia) Pty Ltd. Erection, dismantling, and moving of shuttering was carried out by two 22 RB crawler cranes.

87. No checking of forms was carried out by the Department of Main Roads. A check system was organized in the yard by which all the basic measurements were checked by a Site Engineer. These were marked off on a check list and initialled by him. The complete form was handed to the foreman for him to sign before ordering concrete. Any measurements found unsatisfactory were noted and the form was not signed by the Engineer until these were corrected. This system was successful. Reinforcement cages were erected on a former ahead of the shutters. Setting of the forms took from 10 to 15 hours for two carpenters and one labourer. The Engineer's check took about 45 minutes.

88. Concrete was supplied from a 1-cu. yd batching plant with a pan mixer and distribution was by a system of monorails running at high level and discharging directly into the units. Vibration was by means of 4-in. immersion vibrators operating from a stationary 350-cu. ft/min Holman compressor. Curing was carried out by placing rotary sprays inside each box with sprays operating on the outside walls under a hessian tent. It was found advantageous to operate the outer sprays on a time clock to economize on water.

89. The units were cast over a 12-ft gauge track. A pair of trolleys with built in jacks lifted the unit from its pedestals and it was towed by a small crawler tractor to the main supply track. Here it was lowered by the jacking trolley onto a pair of fixed trolleys on which it was moved down to a small wharf. At this wharf an 80-ft pair of specially-designed sheerlegs built of 39-in. dia. steel tubes ½ in. thick lifted the unit and lowered it to a punt for transfer to the main bridge site. The hoisting was by an 8-ton winch through an 8-part tackle, and luffing was by a 5-ton winch through a 6-part tackle. The anchorage to the sheerlegs was provided by approximately 240 ton of concrete kentledge.

90. For the first rib, lifting was by two trunnions bolted to the side of the unit, but this method proved cumbersome and for the remaining ribs, sections of 5-in. pipe were cast into the units and trunnion pins were fitted into these for lifting and lowering. The trunnion holes were cast 3 in. off-centre which enabled the unit to be turned to its true position with minimum difficulty at the bridge site. Diaphragm handling presented little difficulty at the casting yards as they were sent up to the bridge site in the horizontal position.

Deck beam casting yards at Drummoyne and Gladesville

91. The two casting yards will be described together as they were never operated at the same time and all plant except the mixer was transferred from one side to the other as required.

92. The beams were cast on concrete beds treated with Lift Slab parting compound. Special end blocks were incorporated in the beds to allow for the full weight of the beam after stressing. The Anchor blocks and diaphragms were precast at an outside yard and set up on the beam beds; the anchor blocks were set on rollers. The beam shutters were arranged in 20-ft panels to span between diaphragms. Shutters were plywood with light structural-steel

stiffeners and bracing. Two sets of shutters were used and each required many repairs and complete refacing at least once. The use of plywood for this work is not recommended.

93. Cable ducts were located by special stirrups and cables were drawn into the ducts after they had been fixed in the reinforcement, but before the second side shutter was placed. Concrete was mixed in a ½-cu. yd batching plant with a pan mixer, and was distributed by a monorail system. Vibration was by electric shutter vibrators for the web of the beam and a 2 in. immersion vibrator for the flanges. Considerable difficulty was found in adjusting the vibration time satisfactorily. It was found that if the shutter vibrators were allowed to continue while the flange concrete was placed, considerable damage to the concrete surfaces took place on the webs.

94. The shutters were stripped 24 h after casting. This meant that the underside of the flange of the beam had to be cured. Curing was carried out with aluminium irrigation pipes supplied with water under high pressure from a pump controlled by a time clock.

95. Stressing of the twelve ½ in. strand cables was carried out by Freyssinet jacks and locally-manufactured pumps. Considerable trouble was experienced with the pumps. Very little trouble was encountered in the actual stressing. One cable was destressed owing to a slip in the male cone. The Department of Main Roads rejected a number of male cones on the ground of lack of fit.

96. The beams were moved onto the bridge by double-acting jacking bogies. Lack of space, particularly on the Drummoyne casting yard, required that these bogies should be as compact as possible. They were arranged with two sets of wheels, the main set with a 10 ft 8 in. gauge to move the beam longitudinally, and the second set with a 2 ft gauge were spring loaded and could be jacked down to lift the beam and move it transversely. Beams were moved to the main track by hand winches and from there up the bridge by the RB either by straightforward towing, or by four part tackle from the RB rope.

Launching of deck beams and deck construction

97. Deck beams were launched up a central track. The launching truss was traversed sideways to the final position and the beam was then lowered into its position on sand boxes (Figs 20 and 21). Adjustments to this procedure were made on the Gladesville approach to allow slewing of the truss and owing to their inaccessibility the parapet beams were skidded sideways to their final position. The launching winch was driven by an hydraulic motor and was capable of a 5-ton rope pull at a speed of approximately 5 ft/min. Lowering was effected by two hydraulic jacks operating on a link and pin system. The jacks were powered by pneumatic accelerators to give a pressure of approximately 9000 lb/sq.in. Traversing was by a pair of pneumatic capstan winches, capable of traversing the loaded truss at about 20 ft/min. Under good conditions it was found possible to launch one beam per day.

98. The truss was launched by a jib and counterweight. A deck beam was used as a counterweight and this was strutted off the bottom chord of the truss. The launching jib had a bridle running to the top chord of the truss and a fall to the counterweight beam. Lifting of the nose of the truss was by a tapered element in the fall wire. The whole mechanism was winched forward to the next span.

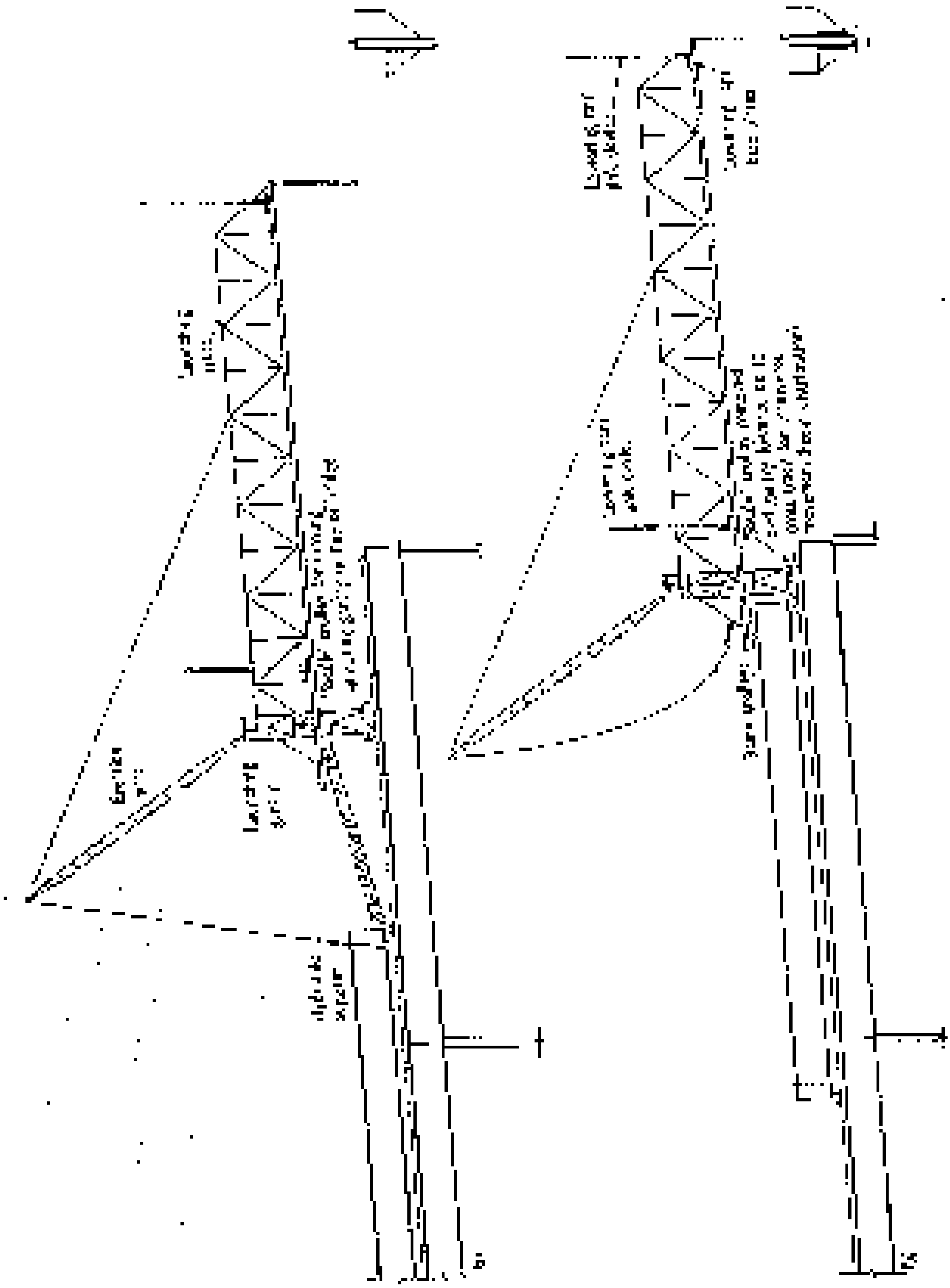


Fig. 20: Launching girder

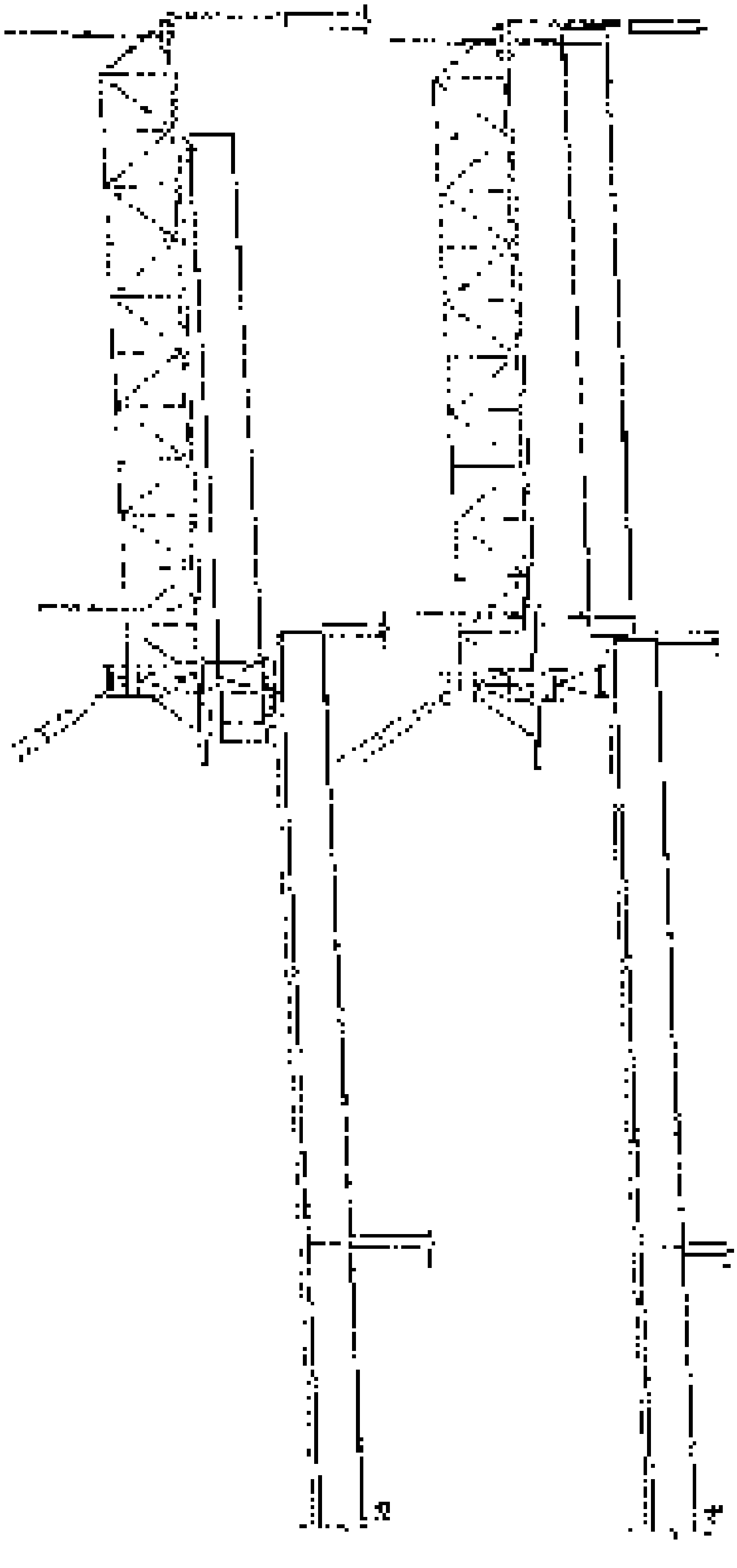


FIG. 30: LAUNCHING GIRDER (continued)

99. The in situ concrete between beams was shuttered from a moving scaffold hung from the beams after launching, and poured from the deck. The main diaphragms were not concreted until all the in situ slab and intermediate diaphragms had been completed on the adjacent spans. In the early stages of the work counterweights were used to compensate for sections, such as the parapet, which had not yet been concreted.

Erection, jointing, and jacking of arch

100. The handling of the arch units was carried out as outlined in §§ 77 to 83 dealing with plant and machinery in the falsework, the cycle of operations for placing a unit being as follows: the unit arrived at the bridge site on a barge in the same attitude as it was cast. It was then picked up by the main 50-ton hoist and turned through 90°. The turning was carried out by arranging the lifting point 5 in. off centre from the centre of gravity. The actual turning was controlled by a chain tackle.

101. The unit was then lifted to the arch level, lowered on to the transporter trolley, and moved upstream to the rib under construction. The appropriate downslope trolley, depending on whether the unit was destined for the Drummoyne or Gladesville side of the arch, was pulled into place under the unit. It was then lifted off the transporter by the downslope trolley jacks. The loaded trolley was moved off the transporter with the 5-ton winch.

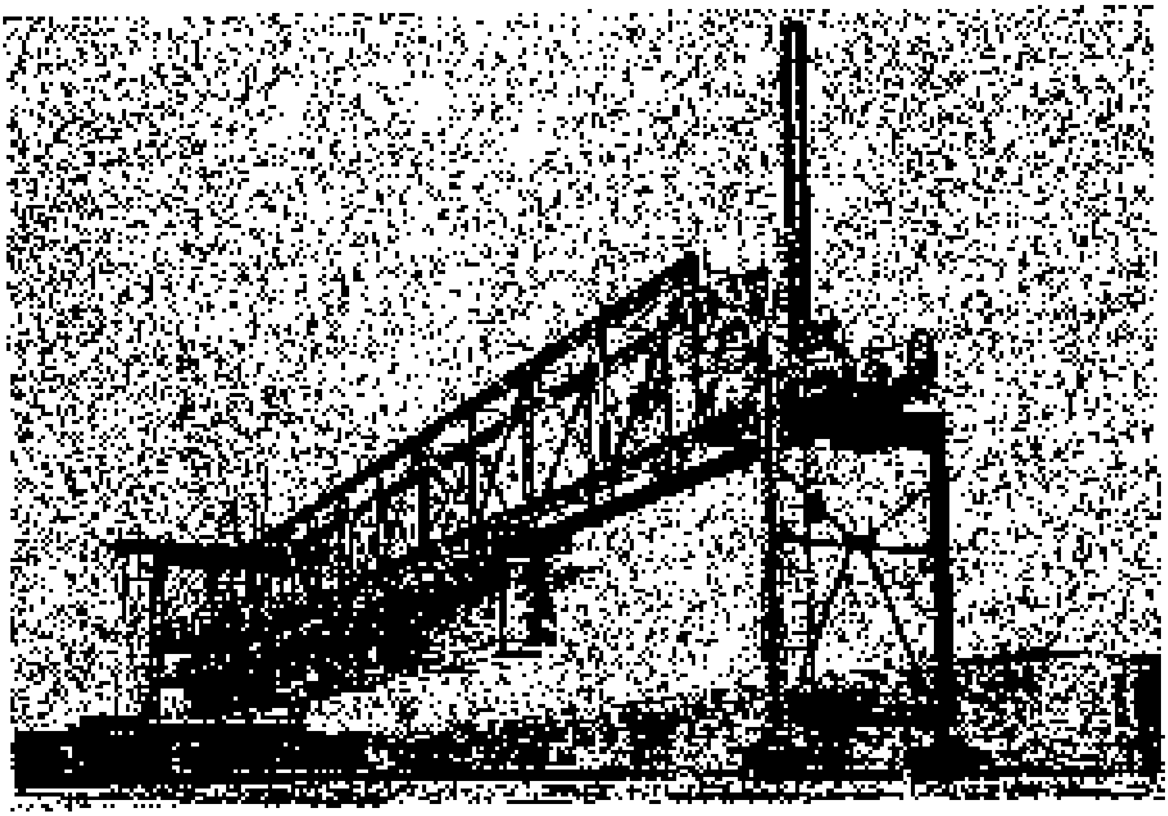


FIG. 21: BEAM LAUNCHING

At the end of the first 50 ft the main 14-ton winch wires were hitched onto the unit and it was ready for lowering down the rib.

102. The arrangements for the diaphragms were similar except that diaphragms were up-ended and not rotated from the punt and while on the two trolleys they were propped. Once the unit was lowered to its position on the arch it was ready for placing in its true position in the rib. Packs to support lift-off the steel members were prepared of a thickness to allow for falsework deflexion. The unit was checked for line by a theodolite from a fixed station on the abutment. The joint thickness was adjusted by timber wedges to give the unit its right location on the arch. As it was found that these wedges moved considerably under load, they were immediately augmented by concrete pads in plastic bags which were cut out once the top and bottom sections of the joint had been concreted. One engineer was in control of each placing team and two rigging labourers operated each trolley. The theodolite station had telephone communication to the trolley position which had a link to the 14-ton winch driver. The winch driver operated as a clearing house for messages from the theodolite points, the trolleys and the 50-ton hoist driver, and advised the central switchboard in the main office of progress, to ensure the supply of units. The average rate of placing was five to six per day, but the best day was 10 units and the best week 43 units in a 6-day week.

103. The jointing followed the units as closely as possible. However, it was a requirement that jointing should not be carried out until at least one clear loaded span of falsework lay between the placing point and the joint being concreted. On the navigation truss this imposed a considerable time lag. The joints were poured in four sections, rubber-faced shutters being used. Concrete was hauled up the ribs in prams by light air winches. Joints were vibrated by 1-in. electric immersion vibrators.

104. Great care was taken with the last joint of a rib. It was desirable to concrete the whole joint at a steady low temperature and then to ensure that during the remainder of the curing period the expansion and contraction of the arch was confined to this one joint. To assist this one face of the joint was painted with parting compound and the concrete was screeded over the surface of the unit to keep the joint clean.

105. In general the behaviour of the falsework under load was as predicted. The settlement of the driven piles on the Drummoyne side of the navigation opening was negligible. Pile settlement in the potted piles at G.3 occurred during the erection of each rib. Remedial action by jacking up the column was taken, but did not appear to be very effective. Compensating allowances were made in setting the units. During the erection of the last rib, settlement occurred at Pier G.5 on the downstream column. This caused rotation of the [illegible] and lateral movement of the rib which could not be corrected at soffit level. Remedial action was taken by jacking up the downstream stanchion of the column with four 10 in. × 18-in. oval jacks, one in each jacking recess. A pressure of 2750 lb/sq. in. was supplied to the flat jacks giving an approximate lift of 600 ton which was sufficient to correct the deflexion.

106. Jacking is covered in another part of this Paper. The procedure was as follows: two engineers were on duty at each jacking point, one in charge of the main circuit pump and trimmer pumps and the top pressure manifold, the other on the falsework under the arch to observe the action of the jacks and to control the second pressure manifold. The main control was in direct

communication with both positions. Measurements of the behaviour of the arch were taken by theodolite and various direct measurement devices and these were relayed directly to the main control by telephone.

107. After difficulties encountered in the first rib a similar time table was followed on the remaining three ribs. Jacking was started on a Sunday evening; at the end of the first stage all jacks were locked off. Next morning,

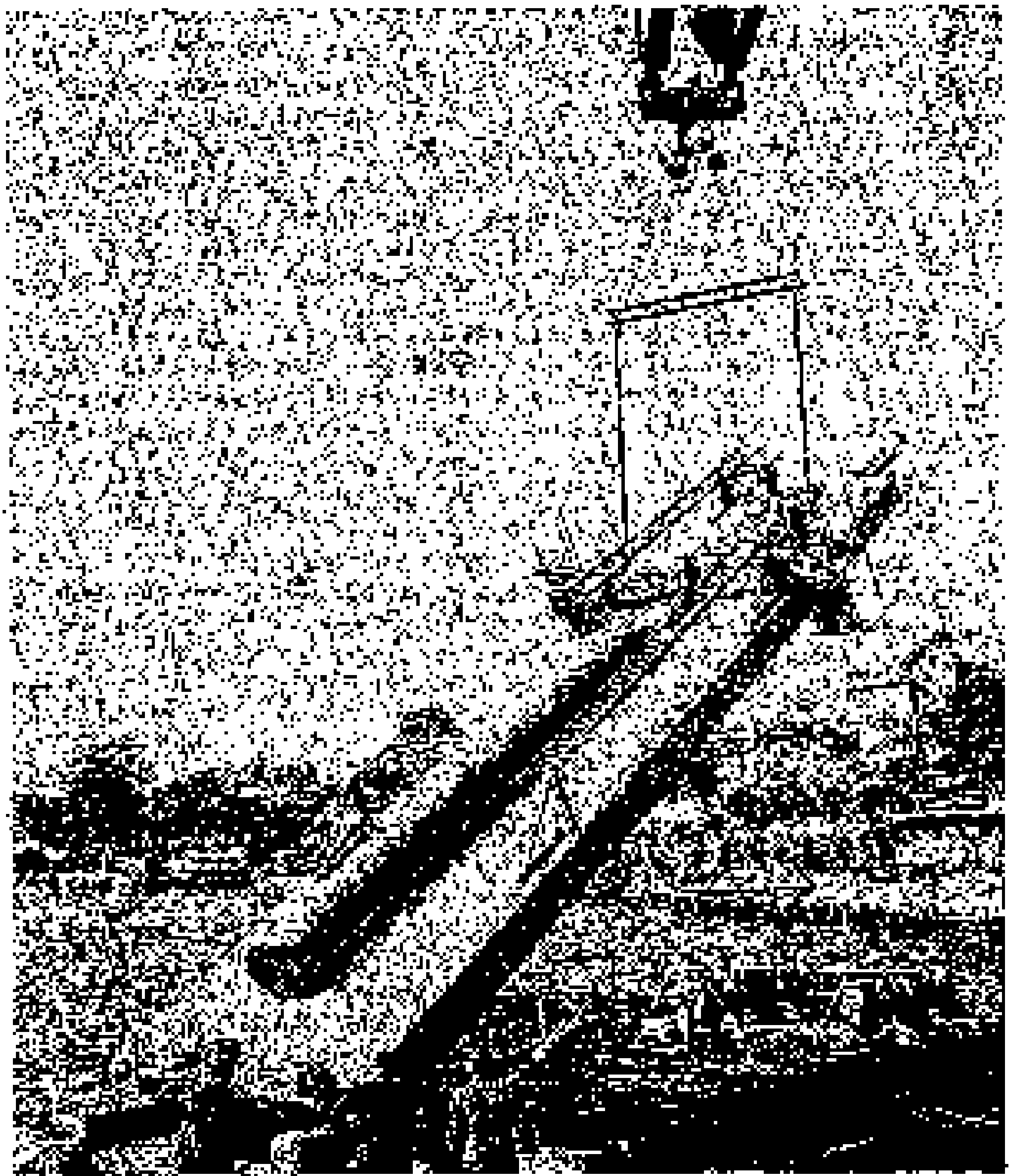


FIG. 22. PRECAST COLUMN WITH PROP ATTACHED

the circuit was removed and the jacks on both jacking points were grouted. The circuitry was replaced for second stage jacking. This was carried out on Tuesday evening followed by grouting on Wednesday. Third stage jacking took place on Thursday evening and grouting of the third stage was completed on Friday. All jacks down to Columns D3 and G3 were released before the third stage jacking started.

108. Some trouble was experienced initially in the replacing of the oil in the flat jacks with grout, in that part of the extension was lost and uneven loading occurred due to uneven losses. This was overcome by pressurizing after grouting with a stressing pump. Observations of the extensions across the jacks were taken with the dial gauges, and four pressure gauges were left connected to four jacks, one adjacent to each corner to observe any alteration in jack pressures as grouting proceeded.

109. After completion of jacking and grouting, the falsework was lifted from its concrete bases, the abutment anchorage destressed, and the whole falsework pulled sideways by lift slab jacks. This was carried out in two sections, each side of the central tower being pulled separately. The columns were then bolted down, concrete footings placed, the anchorages stressed in their new position, and work on the new rib was ready to start. The time taken to move a section of falsework was about one day, the distance moved was 21 ft and the best time recorded was 5 h.

Completion and erection of spandrel columns

110. The work remaining after the jacking of the fourth rib was the jointing, transverse and longitudinal stressing of the arch, the erection of the spandrel columns and crossheads, completion of the deck and the work at the crown of the arch, and dismantling of the falsework. The jointing of the four ribs presented no special problems. Concreting was as for the arch rib joints and transverse stressing as for the deck beams, except that single-end stressing was used.

111. It had been intended to build the spandrel columns in the same way as the approach columns. However, after the experience with the use of the floating crane *Titan* it was decided to precast all columns and crossheads for the arch (Figs 22 and 23). This was done at the Woolwich casting yard, where special beds were prepared. All members were cast on the flat, treated with a slip parting compound, and used as a bed for the next member. In this way, the four columns for piers 5 and 10 were cast in a stack. The four columns for piers 6 and 9 were cast on a second stack, and the four crossheads for these piers on a third stack. Finally after these piers were erected the crossheads for piers 7 and 8 were cast on the third bed.

112. A selected number of the Macalloy bars in the column units were [illegible] for erection, destressed before coupling to the bars placed in the unit, and restressed from the top of the column. The crossheads had additional [illegible] placed over the points of support to prevent cracking when lying flat.

113. Erection of all these members was straightforward except for piers [illegible] and 8, which owing to the interference between the lower chord of the *Titan's* jib and the top surface of the arch, had to be pulled sideways 10 ft on rails.

The weights involved were 80 ton for columns 5 and 10, 35 ton for columns 6 and 9, and 108 ton for the crossheads.

114. The launching of beams and deck proceeded as for the approaches, except that as a matter of convenience some of the deck beams for the Drummoyne spans were cast in the Gladesville yard and these were transferred across the river by the *Titan* (Fig. 24).

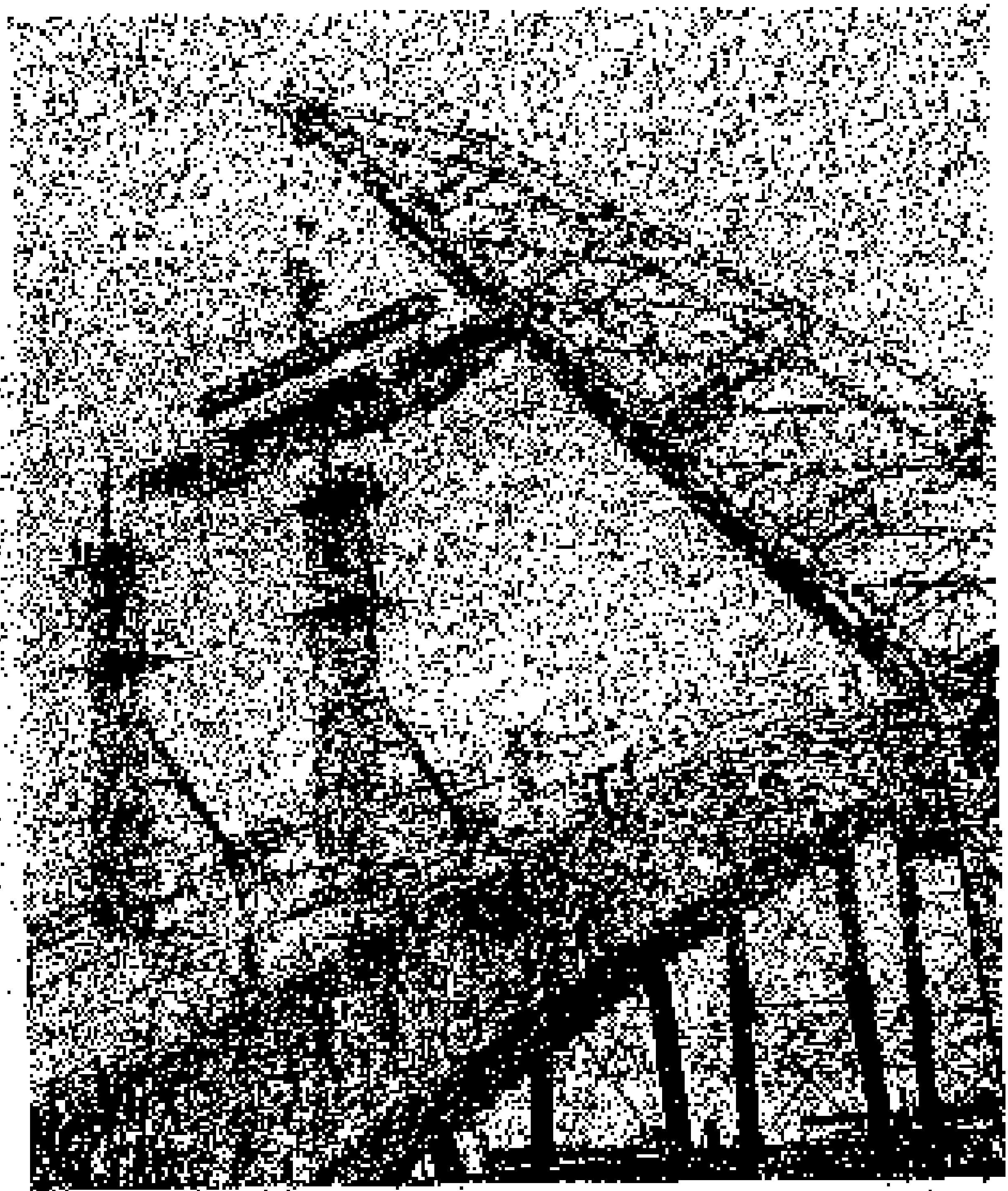


FIG. 23.—*TITAN* PLACING PRECAST CROSSHEAD

115. The work on the crown of the arch consisted of wall and slab construction which merit little comment from the construction point of view.

116. The dismantling of the falsework was carried out by supporting the various members from the arch rib and lowering the columns into the water, then lowering the spans on to punts.

Finishing works

117. Owing to late changes in the kerb, handrail and footways, most of

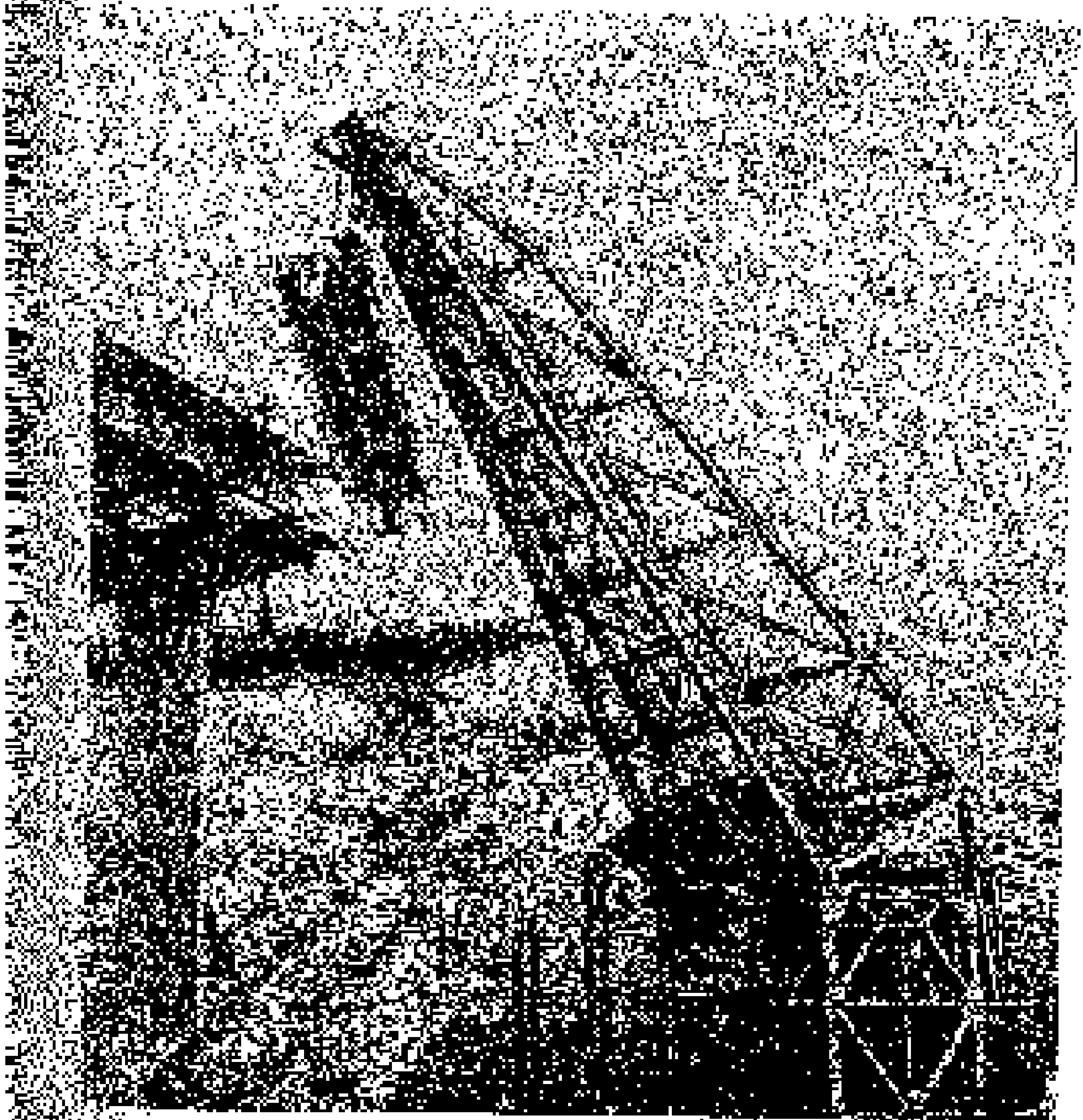


FIG. 24. TRUCK PLACING DECK BEAM

the finishing work on the deck remained to be done after major construction was complete. The only part which continued unaltered was the cantilever parapet. It was found possible to carry out the whole of this work which consisted of approximately 3900 ft of parapet with a single 20-ft shutter by stripping after 24 hours. In the later stages an additional 10-ft wooden shutter was used for filling in short sections.

118. It was finally decided to use the Californian type of kerb. As this is rather complicated in shape, steel shutters were used. Four sets of shutters each 20 ft long were made and with these at least 400 ft of kerb per week was poured.

119. The handrail is of unusual design. The top rail is supported without stanchions by the railings. It consists of a steel pipe of 10-in. nominal bore pressed to form an ellipse.

120. The footway slabs were concreted on edge in batteries, each slab having a shutter top and bottom. A non-slip surface was obtained by using chequer plate for the shutter face on the top side.

ACKNOWLEDGEMENTS

121. This Paper is published by the kind permission of Mr J. A. L. Shaw, *D.S.O.*, B.E., M.I.E.Aust., Commissioner for Main Roads, New South Wales, and of Mr W. E. Reed, M.I.C.E., Chairman of Reed & Mallik Ltd. Mr Shaw succeeded Mr H. M. Sherrard, *C.B.E.*, M.C.E., M.I.C.E., as Commissioner for Main Roads in 1962.

122. The Contractors were Stuart Bros & Partner of Sydney—a consortium consisting of Reed & Mallik Ltd, of Salisbury, England, and Stuart Bros Pty Ltd, of Sydney.

123. R.S.C. Equipment Ltd, of Iver, England were Sub-Contractors for the jacking of the arch ribs.

124. The Department's Resident Engineer was Mr E. A. McKenzie, B.E. (Syd.), A.M.I.E.Aust. The Contractors' work was planned and controlled from the Salisbury office of the Partnership, under the direction of Mr R. H. Kirkpatrick, B.Sc., A.M.I.C.E., Chief Engineer and Director, Reed & Mallik Ltd. The Contract was initially managed by Mr R. A. Voss who unfortunately died in September, 1962, when the management was taken over by Mr W. F. Noelke, Mr M. B. James being the Chief Civil Engineer on the site.

125. The initial and final designs were prepared for the Contractors by G. Maunsell & Partners of London and Melbourne who designed the falsework also. The design was conceived by the late Mr G. A. Maunsell and was worked out in detail by Mr A. F. Gee. Mr Maunsell retained the services of Mr E. Ingersley, A.M.I.C.E., and Mr W. Teskowski, A.M.I.C.E., to assist in the calculations and prepare drawings during the early stages of the initial tender design of the smaller span.

126. During the assessment of the design the Department of Main Roads was advised by Sydney University and Société Technique pour l'Utilisation de la Précontrainte. The Consulting Engineers were advised by Professor A. J. Sutton Pippard, *M.B.E.*, D.Sc., F.R.S., M.I.C.E.

The Paper, which was received on 31 July, 1964, is accompanied by 32 photographs and 16 sheets of drawings, from which the Figures in the text have been selected.

Attachment D

Gladesville Bridge at 50

Gladesville Bridge at 50

http://cenews.com/article/8682/gladesville-bridge-at-50

January 2012 » Columns » STRUCTURES

Anthony F. Gee, P.E.

The Gladesville Bridge in New South Wales, the first 1,000-foot concrete span, built nearly 50 years ago

David Goodyear's excellent article on the spectacular Hoover Dam Bypass Bridge in the August issue of **Structural Engineer** served to remind me how few large concrete arches are built these days. In fact, there have only been six such spans in excess of 1,000 feet since the Gladesville Bridge in New South Wales, the first 1,000-foot concrete span, was built nearly 50 years ago.

The bridge spans the Parramatta River between Gladesville and Drumoyne, west of Sydney, New South Wales, and is the first crossing upstream of the famous Harbour Bridge.

The contract was awarded in October 1957 on the basis of an alternative design submitted by a bidder as allowed by the tender documents. This, and the ground-breaking nature of the design, served to prolong the detailed design and approvals process, and construction did not begin until some three years later. However, the bridge was substantially complete by 1962, although it was not opened to traffic for another two years, pending the completion of the adjacent Tarban Creek Bridge, a 300-foot span concrete portal frame/arch designed and built by the same team.

The Gladesville Bridge is comparatively little known in America, probably because it was designed in England and built in Australia, but at the time it was a noteworthy engineering achievement, setting several new standards and containing a number of innovations in bridge design and construction that have subsequently been widely adopted.

- It was the first 1,000-foot span concrete bridge. The recent development of cable-stayed concrete bridges has resulted in this being almost commonplace but the first big cable-stayed concrete bridge, the Ricardo Morandi-designed Maracaibo Bridge in Venezuela, with spans of 770 feet, was an almost exact contemporary of Gladesville.
- It held the distinction of being the longest span concrete arch bridge in the world for nearly 20 years, until the completion of the Krk Bridge in Croatia in 1980. After nearly 50 years, it still ranks seventh.
- It was the first major concrete arch bridge built using precast segments, albeit with (unreinforced) cast-in-place joints.
- It was the first major concrete arch bridge jacked at the quarter-points, rather than at the crown. Moreover, it was only jacked once, during construction, whereas the Krk Bridge, for example, was re-jacked twice more in subsequent years after completion.
- It was one of the earliest concrete bridges in which the deck was made structurally continuous for live load by the use of unstressed reinforcement contained in cast-in-place concrete between the precast girders over the piers.
- It was probably the first major bridge to rely entirely on the flexibility of concrete columns to accommodate longitudinal movements of the deck due to shrinkage, creep and temperature variation. The 2,000-foot-long deck contains only two expansion joints and is monolithic with the top of each pier. Concrete hinges were incorporated to increase the effective slenderness ratio of the columns, and hence their flexibility, and to prevent moments being transmitted into the deck by the flexure of the piers. The columns have a solid rectangular section only 2 feet wide so that the tallest ones, being approximately 110 feet high, have an effective slenderness (kl/r) ratio in excess of 133.
- It was almost certainly the first bridge to use piers of this kind constructed from precast segments.
- It was also one of the first, if not the first, major bridge to use integral abutments, later to become a popular method of eliminating the problems surrounding bearings and expansion joints at conventional abutments.
- It was undoubtedly one of the first bridges for which the design utilized a suite of computer programs for analysis and detailed design. These programs were written for the purpose, as there was no such thing as proprietary engineering software on the market at that time.

The first program took equations of the parabolas of the top and bottom surfaces of the arch derived from preliminary calculations and determined the elevation of the centerline (the geometric profile) and the true depth, i.e., normal to the profile, of the arch at a large number of stations along its length.

The second program calculated the section properties at each of these stations using the true depth of the section. (The depth of the section was the only variable dimension.) The program then calculated the weight of each segment of the arch between these stations and derived the funicular profile.

The third program performed an elastic analysis of a fixed-ended arch of varying section. It produced influence lines from which the bending moments and thrusts due to the concentrated loads from the deck could be determined and it also calculated the bending moments due to unit thrust and bending moments and thrust due to unit strain so that the effects of rib-shortening, creep and shrinkage could be derived.

The fourth program combined the outputs of the two previous programs to obtain concrete stresses at all sections along the arch. This information was then used to go back and adjust the curves of the

top and bottom surfaces of the arch as necessary, firstly to ensure that there was satisfactory concurrence between the geometric profile and the funicular profile so that it could be assumed that there were no bending stresses due to the self-weight of the arch and, secondly, that there was acceptable residual compressive stress at all sections along the arch under the combined effects of dead and live load from the deck, rib-shortening, shrinkage and creep. In the end, only one iteration was needed to achieve a maximum deviation of 3/8-inch between the geometric and funicular profiles over the 1,000-foot length of the arch.

Finally, the last program calculated the detailed dimensions for casting each of the tapering box-section arch segments, assuming them to be cast upright with their longer face at the bottom (there was no AutoCAD to do this for you in those days!).

Bearing in mind that this bridge was designed in the late 1950s, it can probably quite justifiably be claimed to have been somewhat ahead of its time in several respects. It is still in service and in excellent condition, partly due no doubt to Sydney's mild climate but also unquestionably because of some of the design innovations described above, aimed at reducing or eliminating future maintenance requirements. It could also be relevant that 99 percent of the concrete in the bridge, from the arch to the post-tensioned deck girders and the post-tensioned columns, is in a state of permanent compression.

The bridge was originally designed to carry six lanes of traffic and two generously wide footways. In the late 1970s traffic volumes decreed that one of the footways be converted into an additional lane and this was done without any structural modifications.

Anthony F. Gee, P.E. *has been responsible for the design and construction of major steel and concrete bridges worldwide and is now Principal of Tony Gee International, based in Tallahassee, Fla. Contact him at tonygee@tonygee.net.*

Attachment E

Articles from:

- **Engineering News Record: *Concrete arches to a world record*, 27 September, 1962;**
- **Engineering: *Record Concrete Span over the Paramatta* (sic*)*, 25 January 1963; and**
- **Civil Engineering and Public Works Review: *Constructing the New Gladesville Bridge,* December 1964.**

A. F. GEE

SEPTEMBER 27, 1962

ENGINEERING NEWS-RECORD

A McGraw-Hill Publication

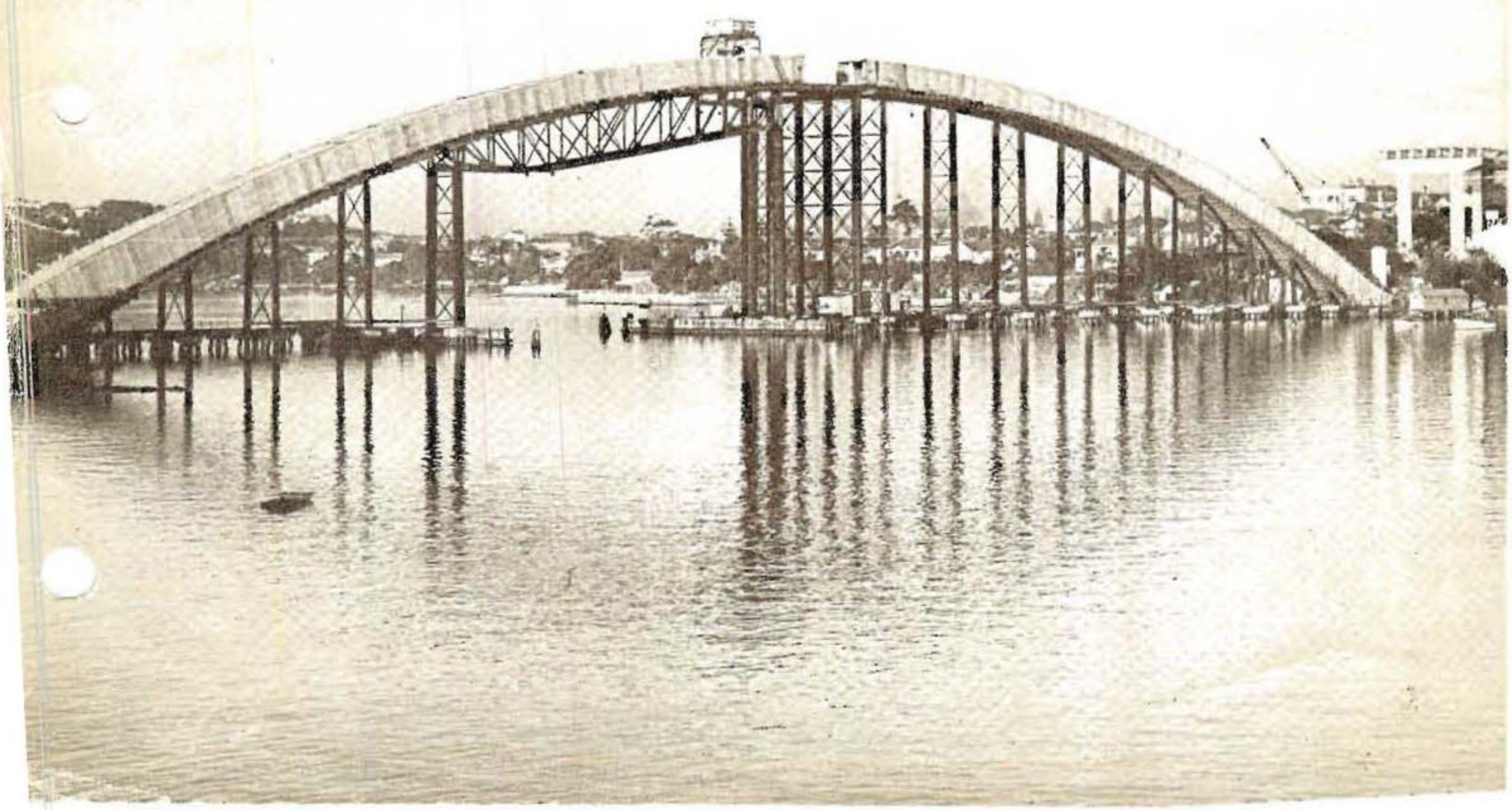
PRICE 50 CENTS

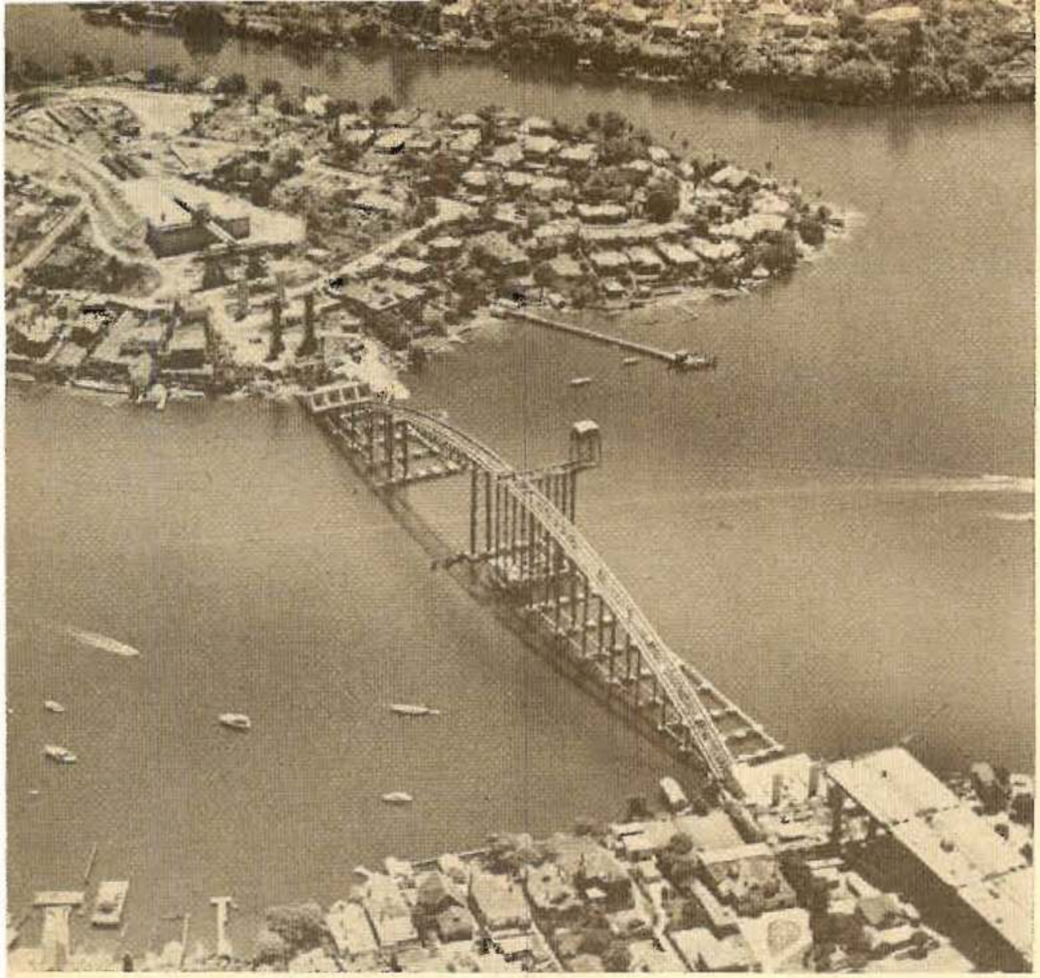
HOTELS CHANGE N.Y. SKYLINE

Hotels, costing more than $227 million and adding 8,000 new rooms, rise in Manhattan p. 38

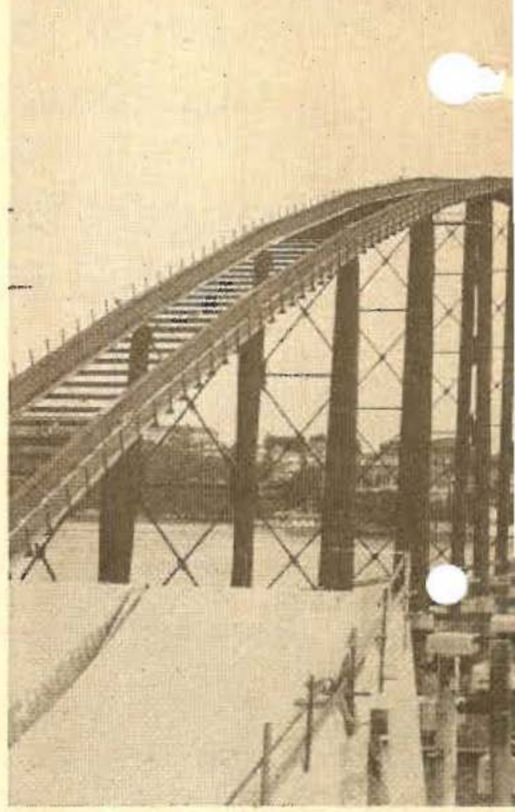
Contents on page 13

Concrete arches to a world record p. 26





Movable centering supports each of four arch ribs in turn.



Hoist lifts precast hollow-box rib

World's Longest Concrete Span

British and Australian engineers and contractors are teamed to design and build the longest (1,000-ft) concrete span in the world over the Parramatta River at Sydney, Australia (ENR Mar. 17, 1960, p. 49).

Design of the graceful arch is based on advanced applications of the theory of plastic design in concrete.

The unique is commonplace and the unusual routine on the Gladesville Bridge. Highlights of the \$6.3-million project include:

• Design of the arch as four separate Voissoir ribs, each comprising 64 pairs of giant precast hollow box units joined in place.

• Construction of each rib on specially designed centering. After each of the first three ribs is completed, the contractor pushes the 130-ft-high centering horizontally to the location of the adjacent rib.

• After all four ribs are finished, transverse stressing will tie the ribs together.

• Installation of slender prestressed columns to carry the approach spans, and the bridge deck over the arch.

The 1,900-ft-long bridge, located only a mile upstream from Sydney's famous steel arch bridge, will carry a new expressway that will be the main traffic artery from the city to its northwest suburbs and beyond. It will have six traffic lanes, with most of the roadway being 72 ft wide, with two 6-ft wide sidewalks. The northern end joins a complex interchange, with the roadway widened to 120 ft at the abutment.

The bridge deck at the crown of the arch will be 150 ft above the water.

• **Design**—Originally, the Department of Main Roads, New South Wales, asked for international bids for a conventional steel bridge of its own design. The original design featured a twin cantilever truss structure with a central span of 740 ft. But the department allowed an option of submission of an alternative design.

A joint venture, Reed and Mallik, Ltd., Salisbury, England, and Stuart Bros. Pty. Ltd., Sydney, picked up the optiou, submitting a bid based on a 910-ft concrete arch. Consulting engineers G. Maunsell and Partners of London and Melbourne designed the structnre for the contractors. They proposed to build it in three sections, with the center section cast on floating falsework.

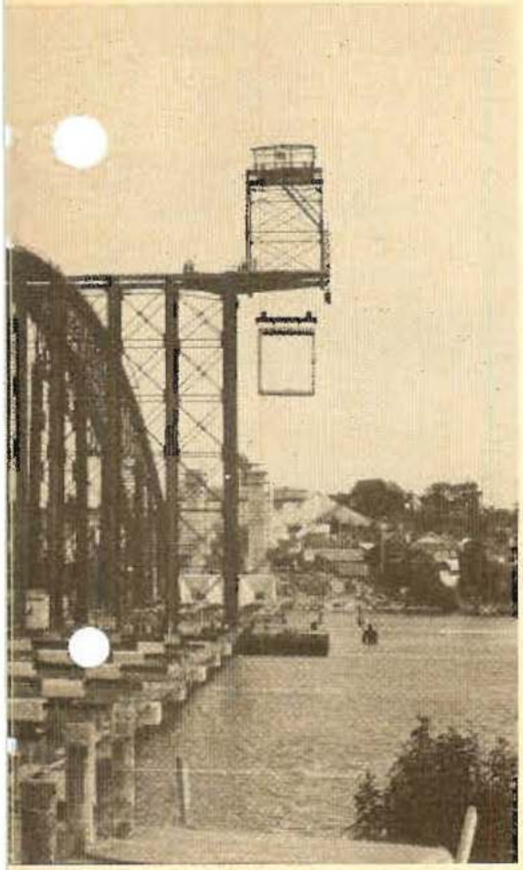
However, Maunsell's design had been based on interpolations of data from borings made for the steel design. Confirmatory borings at the site showed that one of the abutments for the 910 ft arch would have cost substantially more than had been anticipated. The decision was then taken to open up the arch to 1,000 ft, this being the cheapest solution.

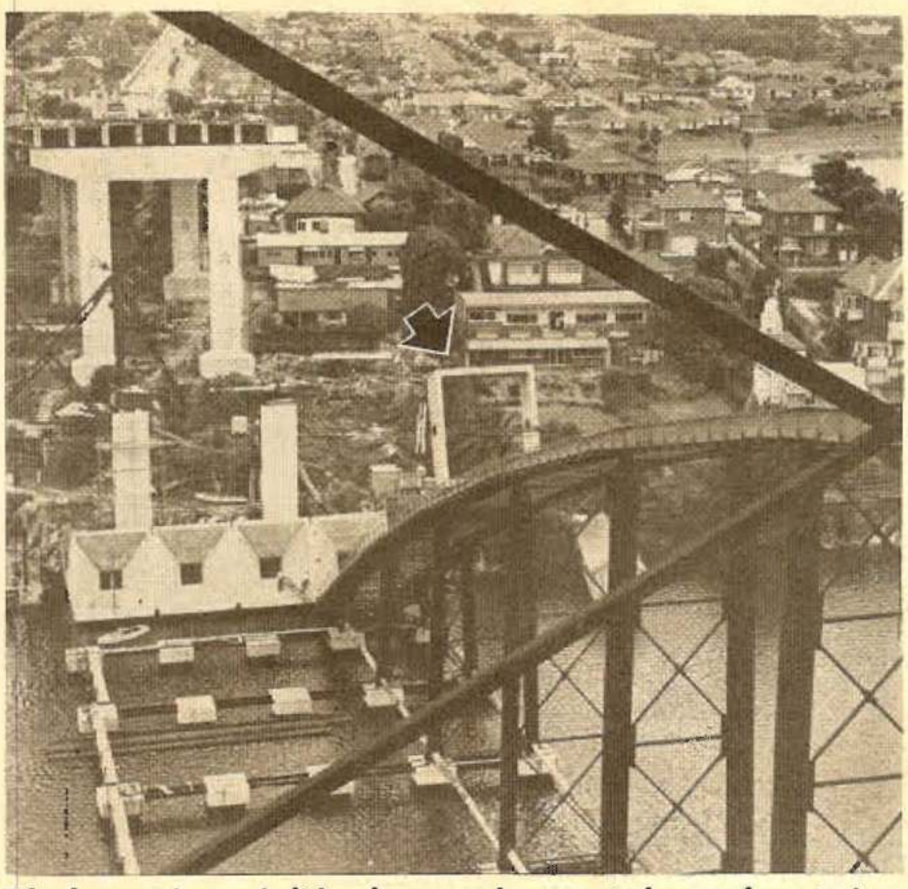
• **Redesign**—Analytically, the jump from 910 ft to 1,000 ft was large enough to force a complete reassessment of the design. It also gave the contractors an opportunity to revise the proposed construction method, ending in the adoption of extensive precasting techniques plus the sliding centering extending across the whole width of the river.

The largest concrete arch bridge in the world now is the Sando Bridge, built in Sweden iu 1942. Its span is 866 ft. The high cost of centeriug extremely large concrete arches has discouraged many designers from using them for major crossings.

Maximum concrete stresses comprise the basic limiting factor in design and

element to top of centering.





Rib element (arrow) slides down tracks mounted on arch centering.

Will Arch Over Sydney Harbor

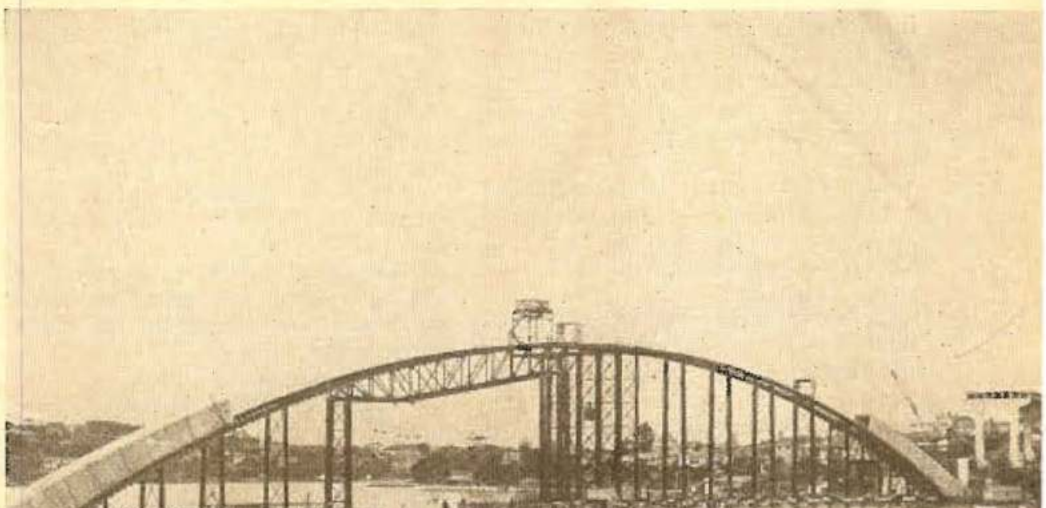
construction. Maunsell's design is based on stresses in the concrete of about 1,500 psi due to axial compression alone, under full dead load plus live load, plus or minus 500 psi bending stresses. The designers believe the construction techniques devised for the Gladesville Bridge have allowed them to approach the ultimate span for a concrete arch.

The revised design was accepted in 1957. However, additional appraisals of the design and construction techniques delayed the start until last summer.

• The ribs—Precast units used in the four arch ribs weigh up to 50 tons, have a common width of 20 ft, vary in length from 7 to 9 ft and have depths ranging from about 23 ft at the spring lines to 14 ft at the crown. Wall and flange thicknesses are constant at 12 and 15 in.

The units are cast in a yard on the river bank, loaded on barges, floated to mid-stream and winched up to the crown of the arch centering. They then roll on trolleys down the arch profile into position; they are carrying the arch up working from both spring lines.

(Please turn the page)



Precast boxes and diaphragms form pure funicular arch rib.

In the springing operation, the crown of the arch is raised about 8 in. above its erected elevation, which is about $2\frac{1}{2}$ in. more than the deflection caused by elastic compression of the falsework under the weight of the rib. The arch is thus jacked above its design profile an amount calculated to compensate for subsequent shortening due to creep and shrinkage.

Designers placed the jacks near the quarter points, rather than at the crown or springlines as has been done in the past, because the points of inflection of the arch for linear strain occur there. As a result, there should be virtually no rotation at the jacking points; this means that all the jacks in a bank can develop their full load under deflection and the reaction will be axial. Function of the trimming jacks is to compensate for any rotation that may occur, so that the main jacks always develop their maximum load, and to modify the position of the thrust if necessary to keep the rib in correct horizontal alignment and profile.

• **Deck and column bents**—The roadway of the structure rests on slender columns founded on the banks of the river and on the arch itself. The columns are generally only 2 ft thick, although the tallest are 110 ft high.

The columns are pinned to a continuous deck structure, must bend to accommodate horizontal deck movements. According to the designers, the slimness of the columns resulted from a need to keep dowu the weight of the superstructure of this large open spandrel arch bridge; to minimize bending moments transmitted to the ribs; and indirectly to eliminate deck expansion bearings and their expensive maintenance.

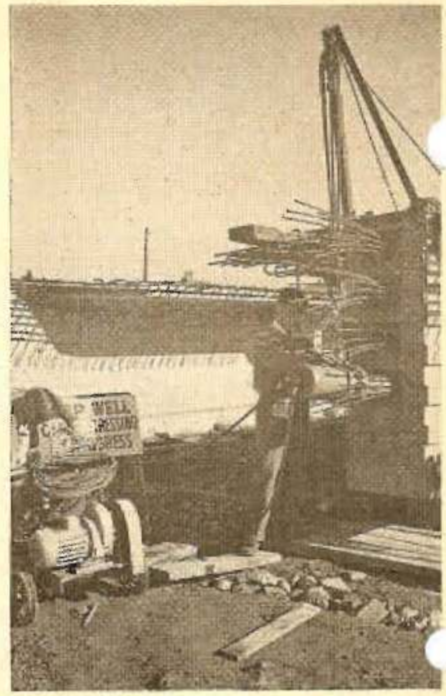
There are 16 deck spans, each 100 ft long. They are carried by eight two-column bents founded on the banks and six bents founded on the arch. A central 300-ft-long deck section rests directly on the arch crown.

The deck is fixed at the abutments and at the crown of the arch, is pinned to the column bents. As a result there are only two sets of expansion bearings in the deck, at the deck quarter points.

The column bents on the river banks rest on spread footings laid directly on sandstone. The longest columns rest on the arch abutments.

Column heights range from 10 ft to 110 ft. All are rectangular in section. All except those under the expansion joints in the deck have the same cross section dimensions, 2 x 10 ft. Columns under the expansion joint are somewhat thicker. Bent cross girders also are 2 x 10 ft.

The designers planned the arch and



FREYSSINET JACKS stress deep girders that comprise roadway deck.

the approaches so that bending stresses in all the columns are about the same, despite the great differences in height. The plate-type design is said to give deep sections to resist lateral wind loads, while still permitting bending in the other direction. At the same time, designers say, the colnmns have adequate resistance to buckling.

The contractor built the approach span bent columns in place, using climbing forms and casting the concrete in lifts about 6 ft high. The columns are prestressed nsing high strength bars and McAlloy techniques. The cross girders are not prestressed, because of problems that wonld be created by the shortening of members during stressing.

Rather than placing the prestressing bars in tubes and later grouting them in place, the contractor wrapped each bar in one layer of a specially impregnated felt and one layer of bitnminous fiber.

The contractor plans to precast and prestress the columns that will rest on the arch. The columns and the precast cross girders will be moved by barge from the casting yard to the bridge, then lifted into place by a floating crane.

Main elements of the deck spans are post-tensioned T-beams 6 ft deep and 100 ft long, with transverse diaphragms. There are eight beams in each span, spaced 10 ft 8 in. c-c.

Part of the deck is made up of the $6\frac{1}{2}$-in.-thick top flanges of the beams. The flanges and diaphragms, which are spaced 20 ft c-c, are made continuous by cast-in-place concrete.

Record Concrete Span Over the Paramatta

Since 1943, the Sandö Bridge in Sweden has had the longest concrete span in the world. It will be surpassed by the end of this year, when Sydney's new Gladesville Bridge over the Paramatta is completed.

SYDNEY HARBOUR has had a famous bridge for many years now, although the massive steel arch was not quite the longest in the world when it was completed. The new Gladesville Bridge across the Paramatta River, an arm of the harbour, between Gladesville and Drummoyne is quite definitely a record-breaker; at 1,000 ft span it will be the longest concrete bridge in the world. Sandö Bridge in Sweden, previously the longest span concrete structure, has an arch 866 ft long.

Fig. 1 shows the stage reached in the construction in September last year. The bridge when complete will consist of an arch of four box-section concrete ribs tied together, supporting a deck of prestressed concrete beams carried on slender prestressed concrete columns. Over the water, the columns are supported on the arch; a central section of the deck 300 ft long is carried directly on the ribs.

Fig. 1 The new and the familiar. Looking over Gladesville's first rib towards the Sydney Harbour Bridge

Traffic Pressure

Fig. 2 shows the reason for the new bridge—the old bridge over the river. This is a two-lane multi-span structure with a swing section on the Drummoyne (sontheast) side. Tidal traffic flow has been used on this bridge for some time now, giving two-lane flow into Sydney in the morning and out of the city in the evening. Unfortunately, ships have priority over road traffic, and if a ship's master wants the swing section opened in the rush-hour, then traffic piles up on the bridge approaches.

In 1957, the Department of Main Roads, New South Wales, invited tenders for a new bridge to span the river. Contractors could either tender for a multi-span steel structure designed by the department (Fig. 3, right) or they could submit other designs on a competitive basis. The contract was won by a tender submitted jointly by Reed and Mallik of Salisbury, Wilts, and Stuart Brothers Pty. Limited of Sydney. The design was prepared for the contractors by G. Maunsell and Partners, consulting engineers, of London and Melbourne.

As originally designed, the bridge had a span slightly under 1,000 ft, based on trial borings carried out by the Department of Main Roads for their own design. Further borings showed that the sandstone rock under the subsoil at the site of one of the proposed abutments was not sound, and moving back the abutment to a more suitable position brought the arch span up to the full 1,000 ft. The successful design, shown in elevation on the left in Fig. 3, has a clear height above high water of 134 ft, with a deck $88\frac{1}{2}$ ft wide. This deck carries 72 ft of carriageway in six lanes, plus two 6 ft wide footpaths. Each arch rib is 20 ft wide by 14 ft deep at the crown and about 23 ft deep at the springings. T[illegible] box sections that make up the ribs weigh [illegible]oximately 50 tons each, with side walls 1 ft thick and top and bottom flanges $1\frac{1}{4}$ ft thick.

Altogether, there are 127 of the sections in each rib, including 19 diaphragms, and the total weight of the arch alone exceeds 25,000 tons.

Fig. 2 An upstream view of the arch showing the old road bridge.

Foundations

Mass concrete abutments have been used for the arch, keyed into the sandstone on either bank. Fig. 4 shows the profile of the thrust block on the Gladesville side; the solid line shows the design profile, and the dotted line the profile finally cut back to solid rock. On this side of the river, the excavations for the abutments were protected by a propped rectangular wooden cofferdam, as the river level is shallow; on the Drummoyne side a semi-circular sheet steel pile cofferdam was chosen, the bank rises here much more steeply. The maximum depth of the excavation on this side exceeded 35 ft below water level.

While this work was proceeding, t[illegible]n-tractors began work on the foundations of the roadway. Reinforced concrete footings on the rock were specified for the main roadway abutments, and also for the foundations of the 10 ft wide by 2 ft thick columns carrying the deck beams. On the northwest side, the Department of Main Roads decided that the bridge should link up with a complex grade-separated intersection, and the originally parallel bridge approach was redesigned to taper from the intersection to the parallel deck over the water

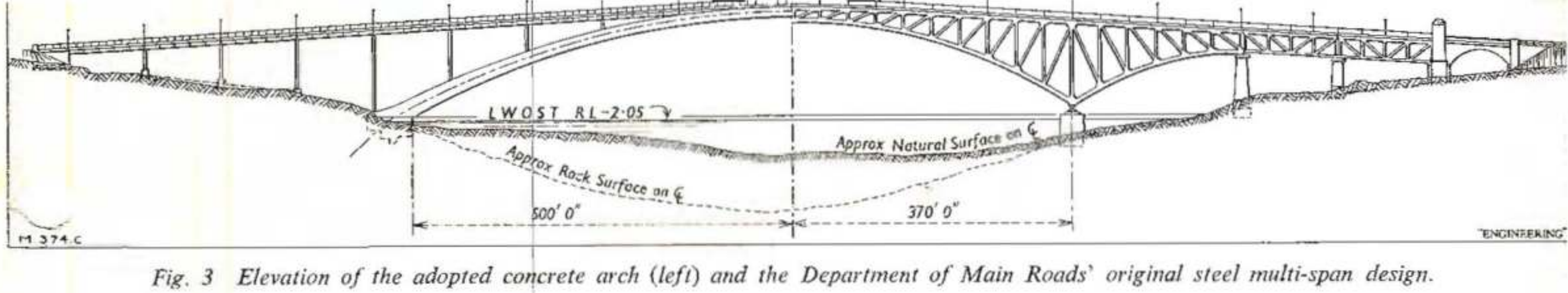


Fig. 3 Elevation of the adopted concrete arch (left) and the Department of Main Roads' original steel multi-span design.

This redesign delayed work somewhat on this side, and the contractors completed most of the Drummoyne approaches before taking their plant to the other bank.

Propped Columns

Each of the columns that support the deck on the land approaches was cast *in situ* on top of the foundations before the Macalloy prestressing bars were tensioned. Since the columns are very slender, they were propped against wind loads in the unstressed condition.

The crossheads at the top of each pair of columns are of reinforced concrete cast in place using prefabricated reinforcement cages. Over the ·ter crossing, the deck support columns and cro...eads will be precast and lifted into place with a 500 ton capacity floating crane. This machine, which played a considerable part in the erection of the centering, can be hired by the hour from the naval dockyard in Sydney Harbour. In Fig. 1, the crane can be seen on the northern side of Cockatoo Island in the middle distance.

Two casting yards were set up on the approach embankments to produce the 100 ft prestressed deck beams that carry the roadway. These beams are stressed with four multi-strand Frev·sinet cables and have four reinforced co· e transverse diaphragms that are later jo·· with *in situ* concrete to complete the transverse section of the deck.

Centering

Tubular steel piles support each of the concrete bases of the centering columns, and these can clearly be seen in Fig. 5. The centering columns are relatively thin-walled tubular steel pillars, cross-braced in pairs. Each pair of columns can be moved sideways as the ribs are completed; the only exception is at the centre of the bridge, where a complete set of columns supports the lif· gear for the precast arch rib units.

Universal section steel joists are supported on top of the columns to carry the arch sections, except over the navigation span, which is offset from the centre of the bridge to line up with the swing section of the existing bridge upstream. The steel beams are supported at the abutments by reinforced concrete corbels, tied back to smaller corbels by Macalloy bars. When the centering is removed, the larger corbels will be cut away, leaving the smaller corbels below water.

Two Scotch derricks were used to erect the first sections of the centering at the abutments, but the remaining sections were prefabricated on shore and hoisted into position with the floating crane. The lattice girder navigation span was also hoisted into position in this way (Fig. 5).

On top of the centering a rail track was mounted, running completely over the arch. A second track runs from the cantilevered hoist (seen in Fig. 5) transversely across the central tower.

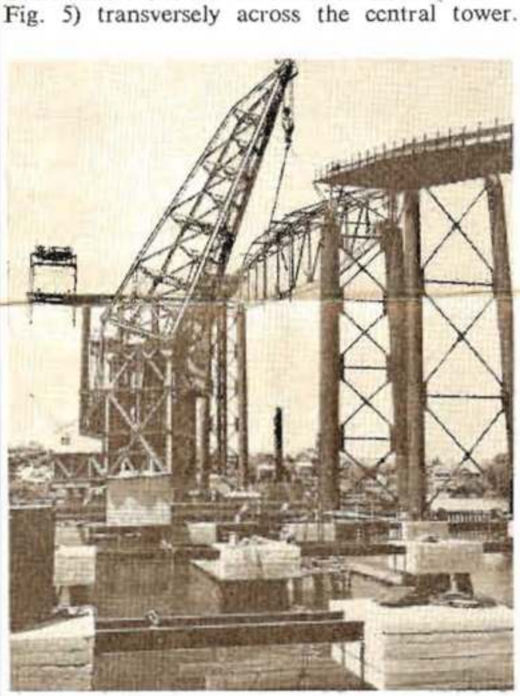


Fig. 5 (above) Erecting the navigation span with the 500 ton floating crane.

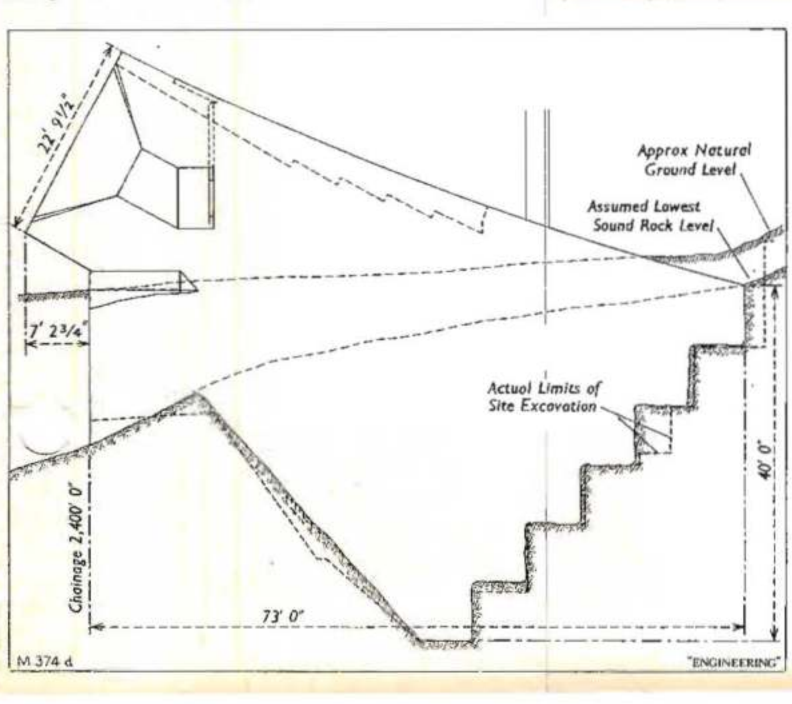


Fig. 4 (left) Profile of the thrust block on the Gladesville bank.

Sections of the arch, cast at a yard on the nort bank of the river almost opposite Cockato Island, are brought up by barge one at a tim hoisted, and dropped on a trolley on the tran verse track. This trolley runs across until reaches the arch track, where hydraulic jack lower the section onto a second trolley. Th second trolley then carries the section down th track until it reaches its final position.

Computing

Deflections of the centering under load wer precisely calculated by the consulting engineer and pre-shaped wooden blocks are laid on th steel beams of the centering to support each un at the right height in relation to its neighbou In the first arch, now complete, it is said that n other adjustment was necessary.

To work out the deflections, and other dimen sions and stresses in the bridge, the consultant made use of a Ferranti "Pegasus" compute Four programmes were written altogether. Th first was used to calculate the influence lines fo a large number of points across the arch, and t calculate the moments resulting from rib-shorten ing due to arch thrust and also from linea distortion due to temperature, shrinkage c jacking. The second programme checked thes properties using a different set of input data. I an arch of the size of Gladesville, the profil must be perfectly funicular under its own weigh and a third programme calculated the exac funicular profile of the arch and compared with the profile derived from the equations to th geometric curves which form the top and bottor surfaces, and which were the basis of the secon programme. Lastly, a programme was writte to calculate the six dimensions necessary t construct each of the 64 different units in eac arch rib. Since the top surface of an arch never parallel to the bottom surface, and th ends of the units have to be normal to the centre line to prevent shear cross the joints (while th 3 in thick joints prevent the front of one un being the same as the back of another) th would have been a mammoth job by han The levels and chainage at each of the four corne of each unit in its erected position were als derived, and the lengths of the units were mad to increase uniformly while maintaining a co stant unit weight.

Casting Yards

To maintain the supply of arch rib units the rate required, the casting yard on the nort bank of the Paramatta is highly mechanize Although the thickness of walls and flanges the units is constant, each unit tapers in bot directions in elevation, and no two units a alike in one half of a rib. Special adjustab shuttering is therefore used, and the sectio are cast on concrete plinths straddling ra tracks.

The units are only accessible in strict rotatio and are cast along the tracks in the order which they are required. Because there is no wa to store finished blocks other than on the castin plinths, any defective unit means a three-wee delay while a new one is cast. So far, only o unit has been rejected, in the early part of th work.

Overhead monorails are used to convey co crete from the batching plant to the shutter

Constructing the New Gladesville Bridge

SPANNING the Parramatta River three miles from its junction with Syduey Harbour, the new Gladesville Bridge replaces a two-lane opening span bridge built in 1881 and forms part of the future North-Western Expressway, which provides for part of the traffic which would otherwise use Sydney Harbour Bridge about four miles to the east.

The bridge, costing some £A.3 million, consists of a four-ribbed concrete arch having a span of 1,000ft and, on each side of the arch, four prestressed concrete girder spans each 100ft long. The thrust from the arch is supported by mass concrete thrust blocks founded on saudstone on either side of the river. The arch has sufficient clearance for shipping 120ft high over a width of 200ft. Each rib is of hollow box sections 20ft wide, with depths increasing from 14ft at the crown to 23ft at the thrust blocks, measured at right angles to the axis of the arch. The clearance between ribs is 12in. Transverse diaphragms at 50ft intervals tie the four ribs together.

The deck system above the arch comprises, over the central [illegible]ft, an in-situ concrete deck slab resting directly on the ribs, and four prestressed concrete girder spans of 100ft on either side of the central portion. The prestressed concrete girders are carried on transverse headstocks by pairs of thin wall-type columns seated directly on the arch ribs at alternate transverse diaphragms. The overall length of the bridge deck is 1,901ft 6in. The width of the carriageway is generally 72ft between kerbs, with a 6ft wide footway on each side.

Each arch rib is desigued as a plaiu concrete voussoir arch consisting of 108 precast hollow concrete blocks and 19 precast concrete diaphragm blocks. Concrete of 6,000 lb/sq.in minimum compressive strength was specified throughout, with a normal working stress of 2,000 lb/sq.in. The precast diaphragm units in adjacent ribs are connected by in-situ concrete to form complete transverse diaphragms across all four ribs. The transverse diaphragms are post-tensioned with cables and supply transverse stiffness to the arch as a whole.

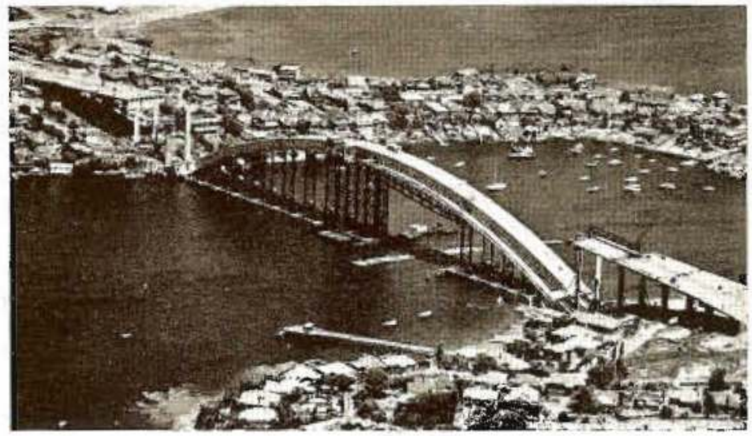
The thrust blocks at the ends of the arch are of mass concrete, and bear on steps cut in the solid sandstone of the river banks. Bearing pressure on the sandstone has been limited to 12 tons/sq.ft with an overload provision of 25 per cent, giving a factor of safety of at least 12 on strength tests of cores cut from the actual rock. The abutments at the two ends of the bridge are of reinforced concrete, and of box-type with earth filling. The side and front walls are designed to resist the load from the filling.

Piers are pairs of prestressed concrete thin-walled type columns. The wall thickness is 2ft except in the tall piers at the ends of the arch where the wall thickness is increased to 2ft 6in. The column bases are widened to form plinths extending some 4ft 6in above natural surface level

Deck girders are prestressed concrete T-beams, eight per span, each with four 12½in strand Freyssinet cables except those on the splayed northern approach which have three 12½in strand Freyssinet cables. A total of 143 girders was required. In-situ concrete is used to connect together the top flanges of the girders, in the end cross girders, and in the connections between the precast intermediate cross girder diaphragms, of which there are four in each span. The whole of the deck expansion is taken at two finger type expansion bearings over piers 4 and 11 respectively at the ends of the arch span.

Preliminary work on the foundations for the arch thrust blocks began in December 1959, with the construction of cofferdams on

Left: Placing pier 5 with a Titan crane. Centre: The precast crosshead being placed for pier 5. Right: Lowering a falsework column, Drummoyne side.



Above left and right: Two views of construction, showing the stages of advance.

Above: The cellular structure of the deck can be clearly seen here. Centre: Lowering the second section of the navigation truss. Below: The bridge nearing completion.

both river banks. After the foundations were excavated they were filled with ready-mixed concrete. The concrete in the arch thrust blocks was placed in layers of about 5ft average thickness and compacted by vibrators.

The abutments are of reinforced Class AA concrete on spread footings on sandstone foundation, the wing walls being almost parallel and at right angles to the front walls. The exposed faces of the wings and front walls are plain with strengthening counterfort ribs behind. Steel pipe falsework supporting lined timber forms was used for forming the abutments.

For the construction of the headstocks the reinforcement was prefabricated on the ground and lifted into position within the forms ready for concreting. For the taller columns, temporary tubular steel braces, anchored to the ground, were attached to the column walls to steady them and prevent their movement under wind load.

The falsework consists of spans of steel beams 60ft long, with a steel truss span 220ft long over a navigation openiug in the northern half of the river crossing, all tied together and anchored at each end to Macalloy bars set in the arch thrust blocks. The falsework was supported on steel tubular columns fouuded on steel tubular pile bents, the piles being taken down to the rock of the river bed.

Lifting gear installed on the central tower lifted the voussoir blocks into position. The tower was designed also to serve as a stay to prevent sideways movements of the individual free-standing arch ribs until they were tied together.

Units for the arch ribs were made at a site on the river shore about three miles from the bridge, where the contractor set up a casting yard. The yard was laid out to accommodate all the units required for one rib at a time. A pan-type mixing plaut was installed with mono-rail delivery from the mixer, to each of the unit positions.

In the manufacture of the deck beams, the contractor precast the end blocks and cross girder diaphragms, set them up in the forms, and then placed the remaining concrete in-situ. To place the deck beams in position, they were loaded on rail bogies and hauled up the earth approach embankment to the abutments, where they were lifted by a special launching truss and placed in position on their seatings.

The contractor's scheme for erecting the arch ribs was to lift the individual blocks from the lighters to the top of the arch falsework and winch them down on bogies to their correct positions on the falsework. In-situ concrete joints were made progressively between the blocks as these were placed until the quarter points were reached, when Freyssiuet flat jacks were installed in four layers, after which the placing aud jointing of the blocks continued until they met at the crown of the arch.

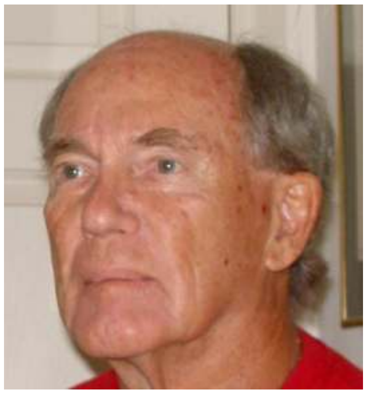
The consulting engineers to the contractors were Messrs. G. Maunsell & Ptnrs. of London and Melbourne. The bridge has been constructed by a partnership of Reed & Mallik Ltd. of Salisbury and Stuart Brothers Pty. Ltd. of Sydney for the Department of Main Roads, N.S.W.

Attachment F

Brief biography of Anthony Gee

Anthony Francis GEE

Born: Oldham, Lancashire, England 16 February 1934



Education:
Malvern College 1947 – 1952
Cambridge University 1953 – 1956

Career:

G Maunsell & Partners	Graduate Engineer / Engineer	1956 – 1963
W S Atkins & Partners	Chief Structural Engineer	1963 – 1968
Mott, Hay & Anderson	Associate Partner, Chief Bridge Engineer	1968 – 1971
Redpath Dorman Long	Chief Engineer	1971 – 1974
Tony Gee and Partners	Founder, Senior Partner	1974 – 1988
Tony Gee + Quandel	Founder, Chairman / Consultant	1982 – 1995
Tony Gee International	Sole Proprietor	1995 – date

Professional Qualifications:

Member, Association of Consulting Engineers (UK)		1978 – 1998
Fellow, Institution of Civil Engineers (UK)	(Member 1959 – 1967)	1967 – 1998
Fellow, Institution of Mechanical Engineers (UK)		1975 – 1998
Fellow, Institution of Structural Engineers (UK)	(Member 1962 – 1972)	1972 – 1998
Fellow, Institution of Highways and Transportation (UK)		1977 – 1998
Licensed Professional Engineer (USA)		1992 - date
Licensed Structural Engineer (USA)		1990 - 2008

Offices:

Member of Council, Institution of Structural Engineers (UK)	1965 – 1968
Member of Council, Institution of Structural Engineers (UK)	1977 – 1980
Founding Member of Council, Steel Construction Institute (UK)	1984 – 1990

Awards:

Telford Premium, Institution of Civil Engineers (UK)	1965
Nominated, Engineering News Record "Man of the Year" (USA)	1982
Oscar Faber Medal, Institution of Structural Engineers (UK)	1987
Telford Gold Medal, Institution of Civil Engineers (UK) (the highest award of the Institution)	1992

Papers and Publications:
Author of more than 30 technical papers published or presented at international conferences.

Attachment G

Eugène Freyssinet (1879-1962)

Eugène Freyssinet (1879-1962)

After Robert Maillart, Eugène Freyssinet (1879-1962) a graduate of the École des Ponts et Chaussées was the second great name in early 20th-century concrete bridge-building.[1]

Appointed bridge and highway engineer at Moulins in 1905, Freyssinet designed and built many reinforced-concrete bridges, including one with a 300-foot (91-metre) span. From the end of World War 1 until 1928 he worked for a contracting firm, and in 1930 he completed the Plougastel Bridge across the Elorn River at Brest. With three 612-foot (187-metre) spans, this was the largest reinforced-concrete bridge constructed up to that time.

Freyssinet pioneered prestressed concrete through his discoveries and tests while researching the effect of creep in concrete. After 1928 Freyssinet devoted himself to the development of pre-stressed concrete and also to the manufacture of high-strength concrete. His most significant discovery was, that only a high-strength steel at a high stress would achieve a permanent pre-stress in concrete.

At first little recognized, Freyssinet's methods were successfully applied at the Gare Maritime (harbour station) at Le Havre, in 1933 and gradually became universally adopted. After his invention in 1938 of a practical tool for applying tension to steel, the use of pre-stressed concrete became worldwide.[2]

In reviewing the design by G. Maunsell & Partners of the Gladesville Bridge in the late 1950's, the NSW Department of Main Roads engaged Freyssinet along with associates Yves Guyon and Jean Muller to advise on the design, which resulted in several amendments. Freyssinet also supplied the bridge joints and flat jacks.[3]

The original design was for a steel cantilever but when Freyssinet was presented with Maunsell's alternative, he recognised the engineer's talent for pushing the boundaries of concrete bridge construction and approved the design.[4]

The most impressive operation in the bridge's construction was the jacking of the massive 1,000-foot arch, the first time such an operation had ever been conducted. This method of raising the bridge off its falsework had been developed by Eugene Freyssinet. He was too ill to make the journey to Australia but his associate Yves Guyon was present.[5]

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[1] David J Brown, *Bridges, A thousand Years of Defying Nature,* Mitchell Beazley UK, ISBN 1845330803 .

[2] Encyclopædia Britannica.

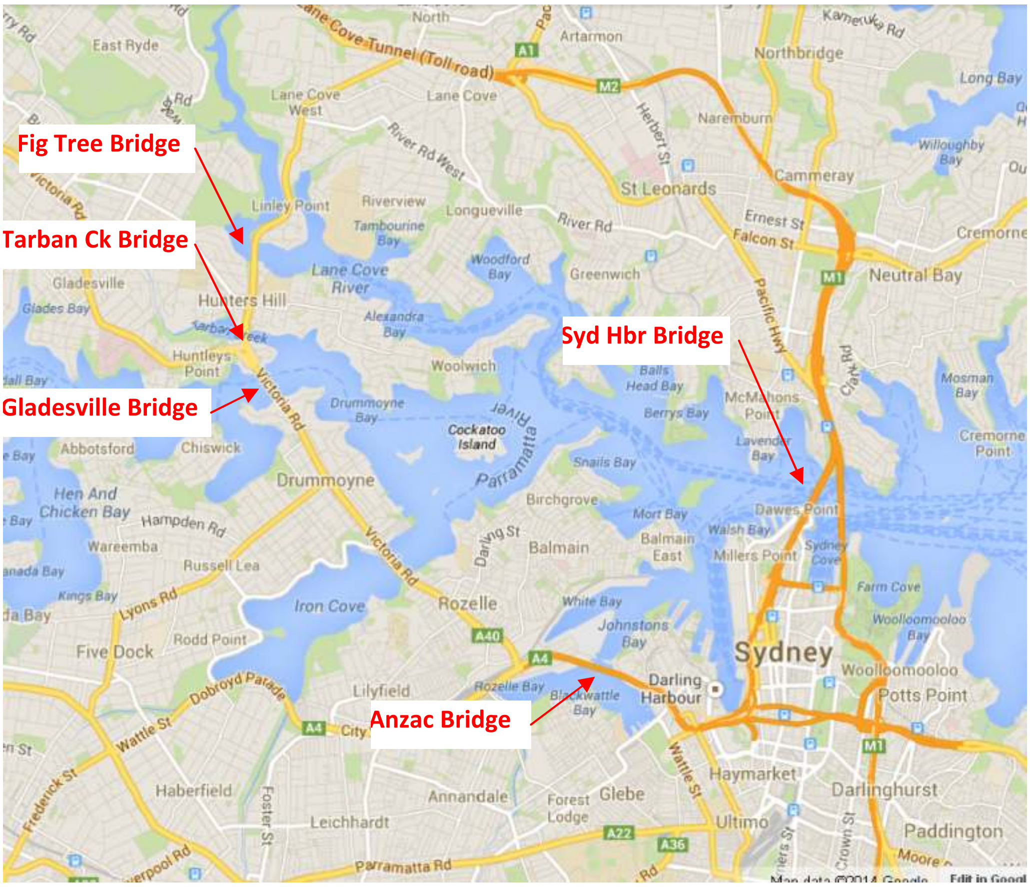
[3] Freyssinet _ Australian Business News Source.htm

[4] http://www.engineering-timelines.com/who/Maunsell_G/maunsellGuy7.asp

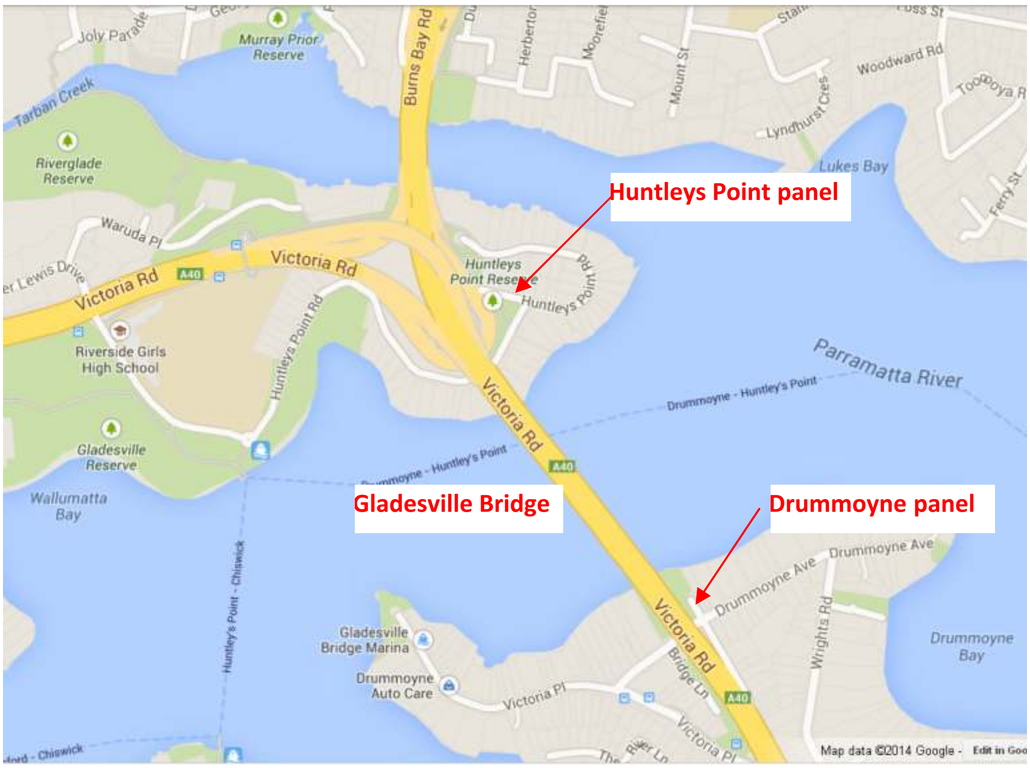
[5] Gladesville Bridge: Notes in support of Engineers Australia *Engineering Landmark* nomination, February 2014.

Attachment H

Maps and Images

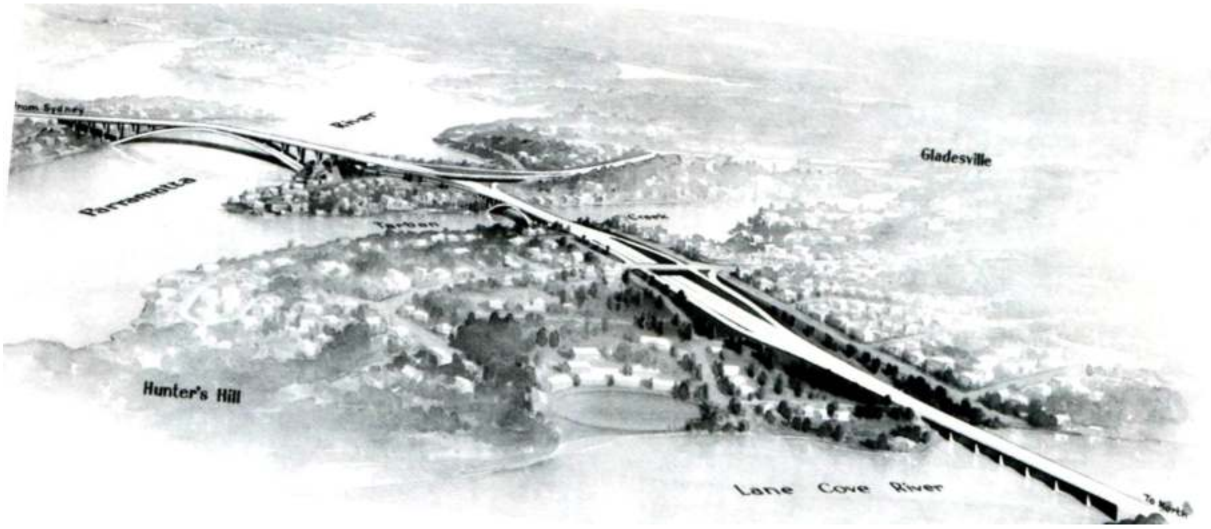


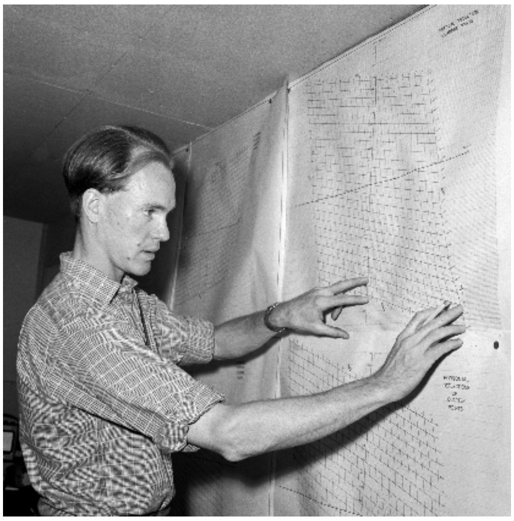
Locality map



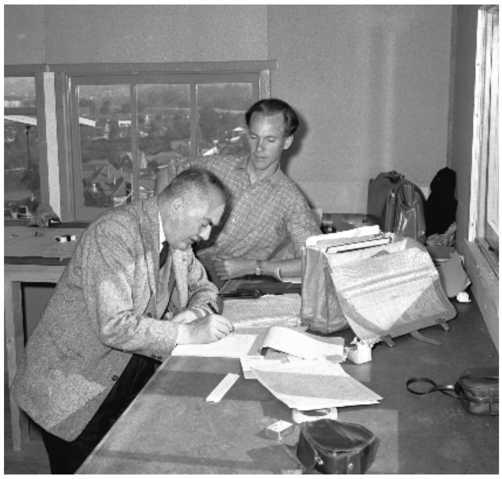
Site map

Proposed North West Freeway showing Gladesville, Tarban Creek & Fig Tree Bridges





Anthony Gee
with nomograms to control jacking



Anthony Gee & Yves Guyon of
Freyssinet's Société Technique pour
l'Utilisation de la Précontrainte

Freysinnet engineer with DMR Asst
Commissioner, Russell Thomas (right)

Arch lifted from falsework. Pat Hills,
Minister for Highways & Jack Shaw (centre),
Commissioner for Main Roads

Huntleys Point Reserve

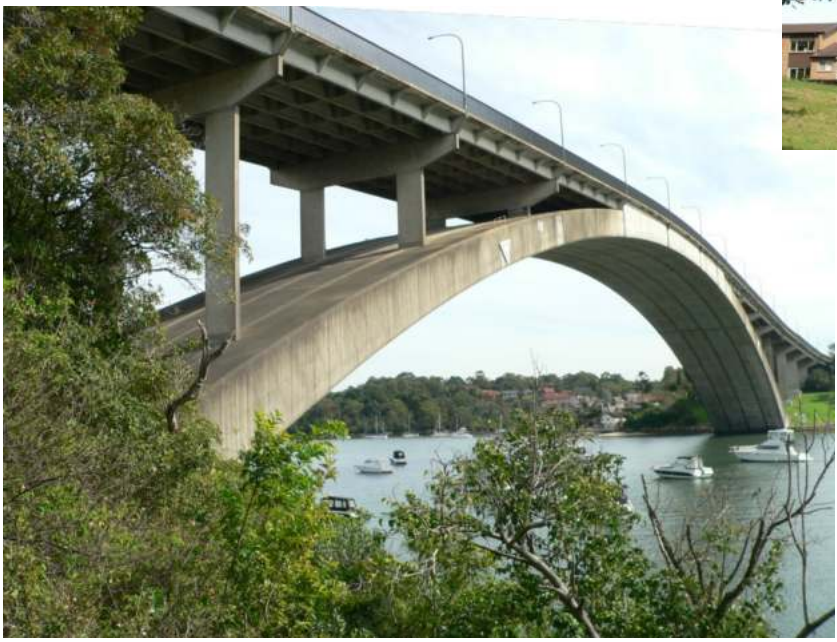
View from interpretative panel, Huntleys Point Reserve

Sydney Harbour Bridge from Huntleys Point interpretive panel

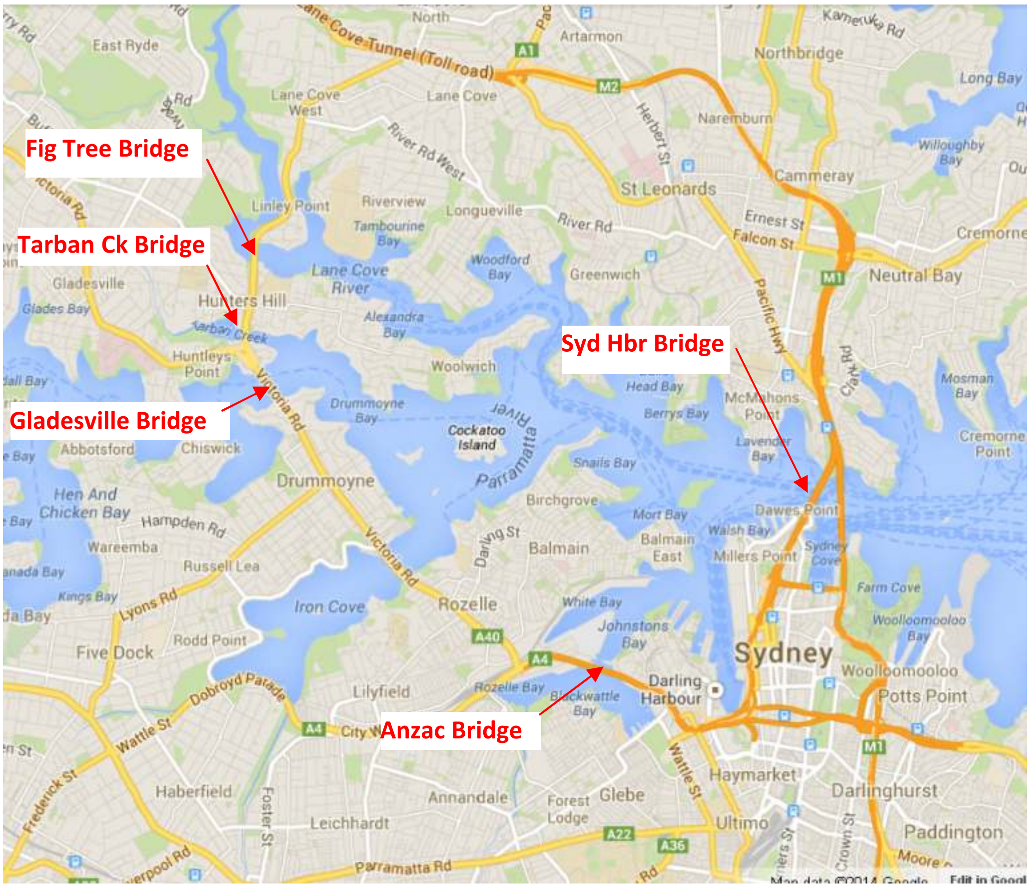
Northern or Huntleys Point abutment

Site of interpretive sign & plaque, Cambridge Road Reserve

Slender columns

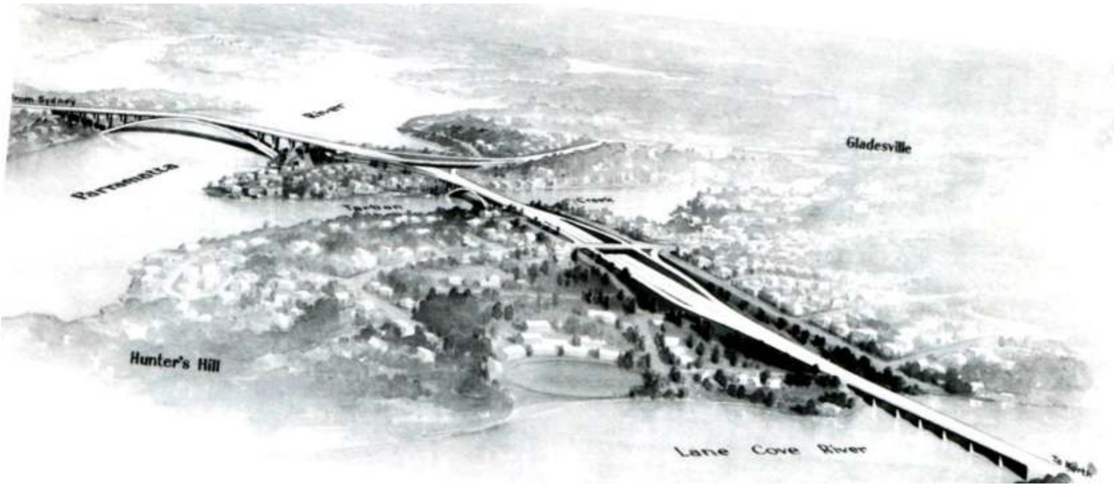


From Cambridge Road Reserve (south east)

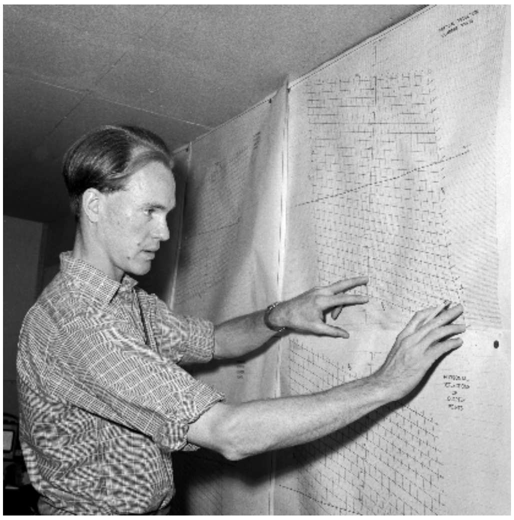


Locality map

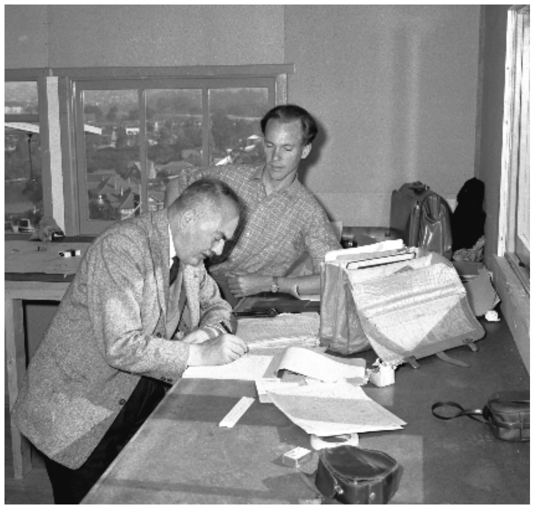
Site map



Proposed North West Freeway showing Gladesville, Tarban Creek & Fig Tree Bridges



**Anthony Gee
with nomograms to control jacking**



Anthony Gee & Yves Guyon of Freyssinet's Société Technique pour l'Utilisation de la Précontrainte

Arch lifted from falsework. Pat Hills, Minister for Highways & Jack Shaw (centre), **Commissioner for Main Roads**

Freysinnet engineer with DMR Asst Commissioner, Russell Thomas (right)

Huntleys Point Reserve

View from interpretative panel, Huntleys Point Reserve

Sydney Harbour Bridge from Huntleys Point interpretive panel

Northern or Huntleys Point abutment

Site of interpretive sign & plaque, Cambridge Road Reserve

Slender columns

From Cambridge Road Reserve (south east)